

University of Southern Queensland
Faculty of Engineering and Surveying

Understanding progressive collapse and its effect to structural design

A dissertation submitted by

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ABSTRACT

The objective of the report is to explore progressive collapse and define some particular terms and review current Building Codes about design methods. Several design methods will be reviewed. It aims to problem solving and work out a best solution for public safety and economical design.

In general, there are two main design principals namely undamaged and damaged design. Undamaged method (i.e. direct method) and damaged method (i.e. alternative load path method) will be generalized, compared and discussed. Also, some particular recommendations are discussed. Finally, a more general discussion on the topic is presented and the report is completed with an overall conclusion.

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Progressive Collapse

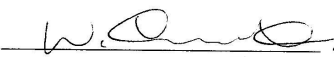
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I further certify that the work is original and has not been previously submitted for assessment in any other course or institution, except where specifically stated.

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Date

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November, 2007

Keen Whale, Kwok

List of Symbols

d	Vertical displacement
x	Horizontal displacement
L	Span of structure
θ	Angle of rotation
M	Mass of object imposed to structure
s	Depth of structure
F_v	Vertical support reaction force
F_H	Catenary force
g	Gravitational acceleration
$\sum F_v$	Summation of vertical force
$\sum F_H$	Summation of horizontal force
$\sum F_M$	Summation of moment
ω	Angle velocity of the mass
f_{vs}	Vertical support reaction when equilibrium
f_{hs}	Horizontal support reaction when equilibrium
$F_{DN15,17}$	Factored axial force for member 15-17
$F_{DN14,16}$	Factored axial force for member 14-16
f_{hd}	Horizontal component at support
f_{vd}	Vertical component at support
T	Centrifugal force of masses
$\%$	Percentage
ND	Design axial load (Factored)
MD	Design moment (Factored)

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1. INTRODUCTION

1.1 Background

This is a report by a final year student of Faculty of Engineering and Surveying (Civil Engineering studies) in The University of Southern Queensland, Australia. The major objective is to investigate the progressive collapse and its effect to structural design. Dr Karu Karunasena, Associate Professor in the University, one of his teaching and research interests being principally in structures and materials, supervised the report.

With respect to the aspect of this report, *‘Understanding progressive collapse and its effect to structural design’*, the report mentions about structural design for civil and building structures such as bridges and buildings (especially large infrastructure, government and military facilities) which have become highly risky recently.

In 1968, one accident took place in UK. That was an explosive accident to induce chain reaction to result of progressive collapse. The spectacular nature of the collapse created an enormous impact on the philosophy of structural design and resulted in important revisions of design codes. That was beginning to consider progressive collapse in the Building Code.

Until recent, terrorist attack on September 11th, 2001 in U.S.A makes public worry about safety of building. Few thousand of people were killed in this attack. Some people stated “current building codes shall be amended to provide protection against progressive collapse caused by extreme case”. On the other hand, some people opposed

by the reason of very high cost in building.

Generally speaking, people recognize that public safety is first priority in building design. Most of people wish all building should be capable to withstand any impact forces such as air crush and explosion etc. But it is impossible because of extreme high construction cost and bulky appearance.

1.2 Objective

Since 911 terrorist attacked on 11 September 2001, the demands by the public to amend current building codes and provide protection against collapse caused by extreme events arise. The objective of the report is problem solving and work out a best solution for public safety and economical design. This report will explore progressive collapse and define some particular terms and review current Building Codes about design method. Several design methods will be reviewed and one of method (alterative load path method) will be discussed in details. Alterative load path method is the most radical and economical solution to problem of progressive collapse. Practical examples from the case study will be analyzed. Furthermore, discussion and research the effect to public safety and economic efficiency will be stated later. Suggestions to be imposed current Building Codes will be recommended. Finally, discussion and conclusion will complete the report.

1.3 Definition

1.3.1 Progressive Collapse

A situation in which a localized failure in a structure, caused by an abnormal load (see paragraph 2.2), triggers a cascade of failure affecting a major portion of the structure and totally collapse. Several buildings have collapsed in this fashion in recent years, and the possibility of progressive collapse is a source of continuing concern. Several alternative methods to deal with the problem of design for the prevention of progressive collapse are reviewed. A computer analysis program capable of tracing the behavior of framed structures through collapse is explained. Of particular note is the capability to remove selectively any member in the structure and determine if collapse will result. Several examples using interactive computer graphics techniques in applying the collapse resistant design procedures are presented. The debris which is derived from part of structure damaged becomes great dead weight to impose remaining structure and induce totally collapse. (American Society of Civil Engineers, 2007)

It has its own characteristics, *disproportionate to initial failure* and *chain reaction of failures*, in its failure mode. Disproportion refers to the ratio of the overall structural collapse to the *initial failure* rather than the ratio of the overall structural collapse to the whole structure. An additional characteristic of progressive collapse is a chain reaction of failures. The collapse of the Ronan Point apartment illustrates well this failure mode. No detailed explanation will be given. However, since the terms ‘disproportion’ and ‘chain reaction’ are subjective, it is very difficult to define disproportion numerically and to allocate certain levels of failure as chain reaction. In later of this report, some guidelines will be quoted and discussed from the Building Regulation 1985.

1.3.2 Abnormal and Accidental Loads

Both are loading event which has a low probability of occurrence to a structure under

normal operation. 'Abnormal load' is defined as a loading event which has a low probability of occurrence to the structure and also is unpredictable. However, there is no certain type of load classified as accidental load or abnormal load. For example, explosive load itself could be either an accidental loading event or abnormal loading event. It all depends on the function of the bearing structure. In general, in the case of a dam structure, the probability of occurrence for an explosion would be extremely low and unlikely. Therefore, explosive load for a dam would be regarded as an abnormal load. On the contrary, in the case of a fuel station, explosion would be more probable and explosive load would, therefore, be regarded as an accidental load. In general, the probability of having an accidental event varies from structure to structure but that of an abnormal event is considered to be the same for all structures. It is example of explosive or impact of vehicle and aircraft. Accident load is same as abnormal load and defined as load event must be caused by accident occur. The probability of particular load event will be extreme low in particular structure but some structures will be higher probability. For example, fuel station is higher probability than footbridge for explosive occurrence. It is necessary to investigate the causes of progressive collapse such as abnormal and accident load. For structural design, the loading is based on possible loading events dependent on the use and nature of building or structure. According to current design codes, the characteristic load that is identified the loading possible occurs. The characteristic load multiplied by safety factor is design load. The structure will be analyzed and designed basis of design load. Nevertheless, progressive collapse will take place in case of abnormal and accident load damage key element (see paragraph 2.3).

1.3.3 Key Element

The element of a structure, for example beam, column, wall, etc., on its losses of load bearing capacity which would lead to a failure of other structural elements. If a building

structure has a redundancy of zero such that it is statically determinate, then it must contain at least one key element. However, structures which have a redundancy greater than zero do NOT imply they have no key element(s). This can be seen from the case study.

1.3.4 Classification of Progressive Collapse

Once accident takes place and collapse occurs, most of people will think about chain reaction failure which is main factor to cause the collapse. Actually, it is not every collapse is progressive collapse. For example, a big bomb was explosive in a small building. The power of the accidental load was so great that it destroyed the whole structure of the building, and one can only say the structural collapse but not progressive collapse has occurred. The so great accidental load is very extreme case and low probability to occur. So, it is impossible to take consideration in structural design for every building.

For another example, when the accident occurred in Ronan Point apartment (See section 3 for detail), there were structural collapse but the collapse elements were not all damaged directly by the accidental load. Since the collapse was out of proportion to its initial failure, it was classified as progressive collapse shown by the red path in **Figure 1.1**. However, if the above collapse is not disproportionate to initial failure but instead there is a chain reaction of failures, it is still regarded as progressive collapse. However, one characteristic is the collapse of the structural elements should not all be damaged directly by the accidental load.

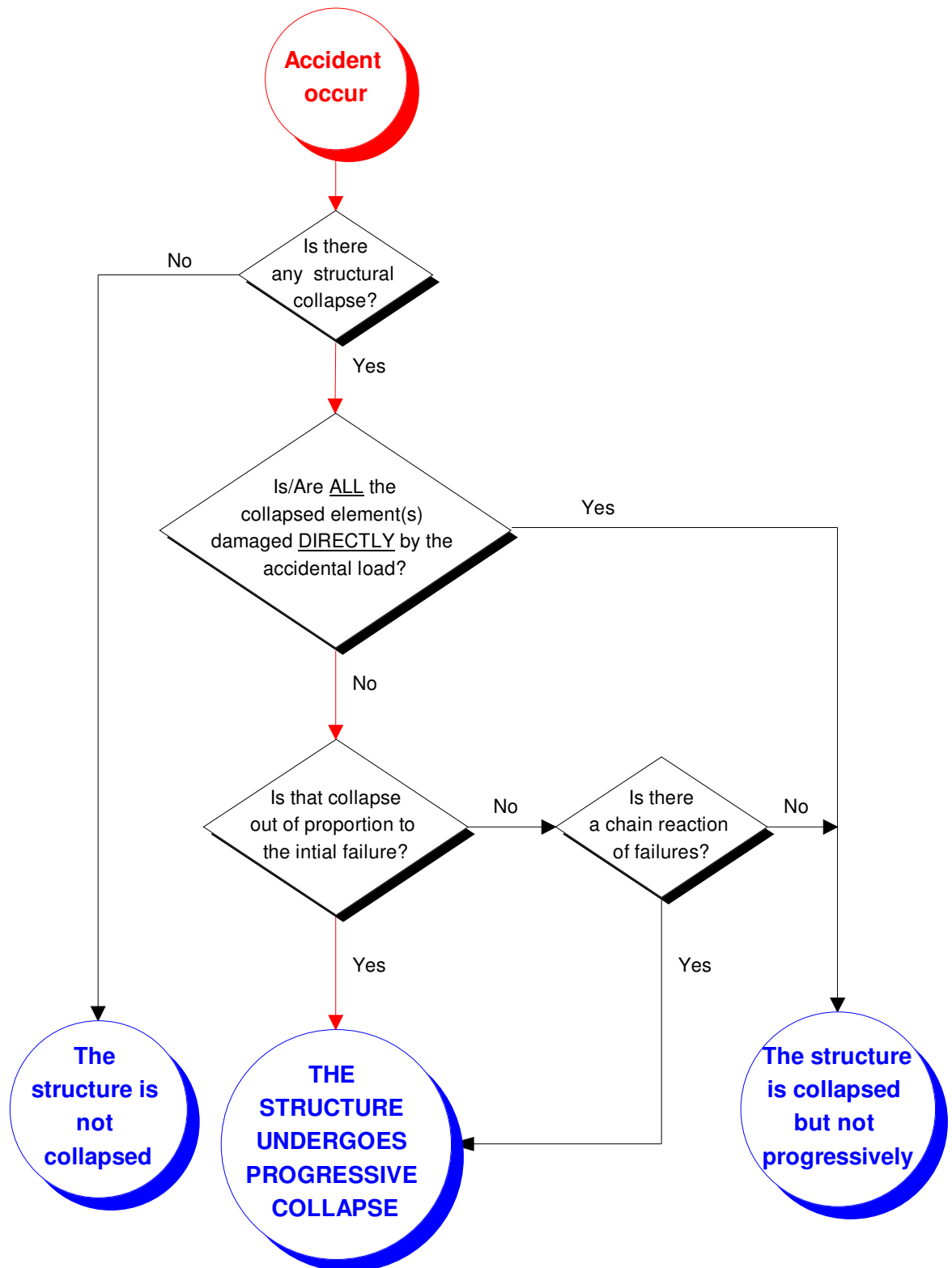


Figure 1.1 - Flow Chart for Identifying Progressive Collapse by Accidental Load

For Figure 1.1, it is a flowchart which is useful for classification of progressive collapse. Progressive collapse should be prevented for some important buildings and civil engineering structure.

If the collapse is either ‘disproportion to its initial failure’ or ‘a chain reaction of failures’, it would be regarded as progressive collapse. It had been said that these two terms were quite objective and not easy to define. However, some guidelines were given in regulation D19 of the ‘Building (Fifth Amendment) Regulations 1970’ In the paragraph 4 and the clause ‘deemed to satisfy’, ‘disproportion collapse’ was confined as the lesser of 750sq. ft. (69.68 m^2) or 15% of floor area of the storey. Also, there was a confinement for ‘chain reaction of failures’ such that the failure was confined within each storey. Besides it was found that the collapse of plan floor area for Ronan Point apartment was neither greater than 69.68 m^2 nor over 15% of the plan floor area of the building.

In next chapter, some past events of progressive collapse will be stated. Most of collapse took place instantaneously and caused many people to lose their life.

2. LITERATURE REVIEW

2.1 Review of Progressive Collapse Case in U.K

Before Building Regulation 1970 (Fifth Amendment) in UK, rare document and research were related to progressive collapse. Besides, no specific requirements or standard provided to guide against progressive collapse.

Since 1968 May, Ronan Point apartment was collapsed caused by natural gas explosion in kitchen where was in one of four corners of 23-storey pre-cast concrete building as shown in **Figure 2.1**. It was trigger for progressive collapse of all the corner units above and below that unit. The spectacular nature of the collapse created an enormous impact on the philosophy of structural design and resulted in important revisions of design codes. The Ronan Point report of the Court of Inquiry stated:

"It is the common aim of structural engineers so to design their structures that if one or two component parts or members fail due to any cause, the remaining structure shall be able to provide alternative paths to resist the loads previously borne by the failed parts." (**Walters Forensic Engineering, 2007**)

Since 1968, the Ministry of Housing and Local Government issued general recommendations, certain circulars and notes after Ronan Point apartment explosive underwent progressive collapse. It was guide to design reinforced concrete (R.C) structure to resistance of progressive collapse. That will be stated later.

“This is a form of "domino effect" failure that can occur in a reinforced concrete structure whereby a failure starting in a particular component rapidly propagates to other components precipitating a major or even a total collapse. The three most common occurrences of this type of collapse are as follows:

1. High rise concrete flat-plate structures (during construction or earthquake).
2. Formwork for concrete structures.
3. High rise structures constructed with precast concrete elements.”

(Walters Forensic Engineering, 2007)



Figure 2.1 - Ronan Point Apartment Building after Collapse

The Ronan Point failure resulted in the addition of an amendment to the British Building Regulations of 1970, later developing into BS Cp 110-1972, which made it mandatory in Britain for buildings of five or more stories to be designed for the possibility of progressive collapse. This amendment applied to all structures of more than five storeys and not limited to designs using precast panels. It followed the alternate path theory and requires, in essence, that every building be designed using either of the following alternatives:

A) The designer shall ensure that the removal of any of the structural components essential to the stability of the building does not produce the total collapse of the structure and that any resulting "local" damage or collapse be restricted to the stories above and below the one at which the removal of the component was made.

B) Structural members shall not collapse if subjected to the combined dead and imposed loads acting simultaneously with a pressure of 5 psi (34.48 kPa) in any direction and any extra loads transmitted from adjacent parts of the structure subjected to this 5 psi pressure.

The need to safeguard against progressive collapse was beginning to be recognized, though not formalized in design codes, even prior to the Ronan Point collapse. The Committee European du Beton produced and published in March 1967 a comprehensive code covering the design and construction of systems buildings. This Code drew attention to the danger of progressive collapse in the following words:

"One can hardly overemphasize the absolute necessity of effectively joining the various components of the structure together in order to obviate any possible tendency for it to behave like a "house of cards"..."(Walters Forensic Engineering, 2007)

In North America the first provisions to deal with progressive collapse were those in the BOCA (Building Officials and Code Administrators) 1981 Code. However, it was not until 1989 that the ACI - 318 Code made a first mention of the problem and not until the issuance of the 1995 Code these provisions comparable to those of BS-CP 110: 1972 were adopted.

The Canadian Standard CSA - A23.3-94 for the design of concrete structures now recognizes structural integrity as a separate limit state. The Standard includes several provisions to enhance structural integrity especially in precast and tilt-up structures, mixed or unusual structural systems and structures subjected to severe loads such as vehicle impact or chemical explosions.

The Ronan Point case stands as one of the few landmark failures that have had a sustained impact on structural thinking, an impact that affected even institutions that traditionally tend to resist change such as the ACI. (Walters Forensic Engineering, 2007)

2.2 Review of Progressive Collapse Case in U.S.A

On September 11, 2001, a terrorist attack took place in U.S.A. Pentagon and Manhattan was struck in a complex and coordinated terrorist operation involving a series of assaults. Islamic terrorists hijacked four jetliners using primitive weapons. The hijackers flew two of the planes into the Twin Tower and third plane into the Pentagon. Passengers on the fourth plane attacked the hijackers causing the plane to crash in Pennsylvania. The towers were weakened by fire and collapsed as shown in **Figure 2.2**. The attack killed nearly 3,000 peoples.

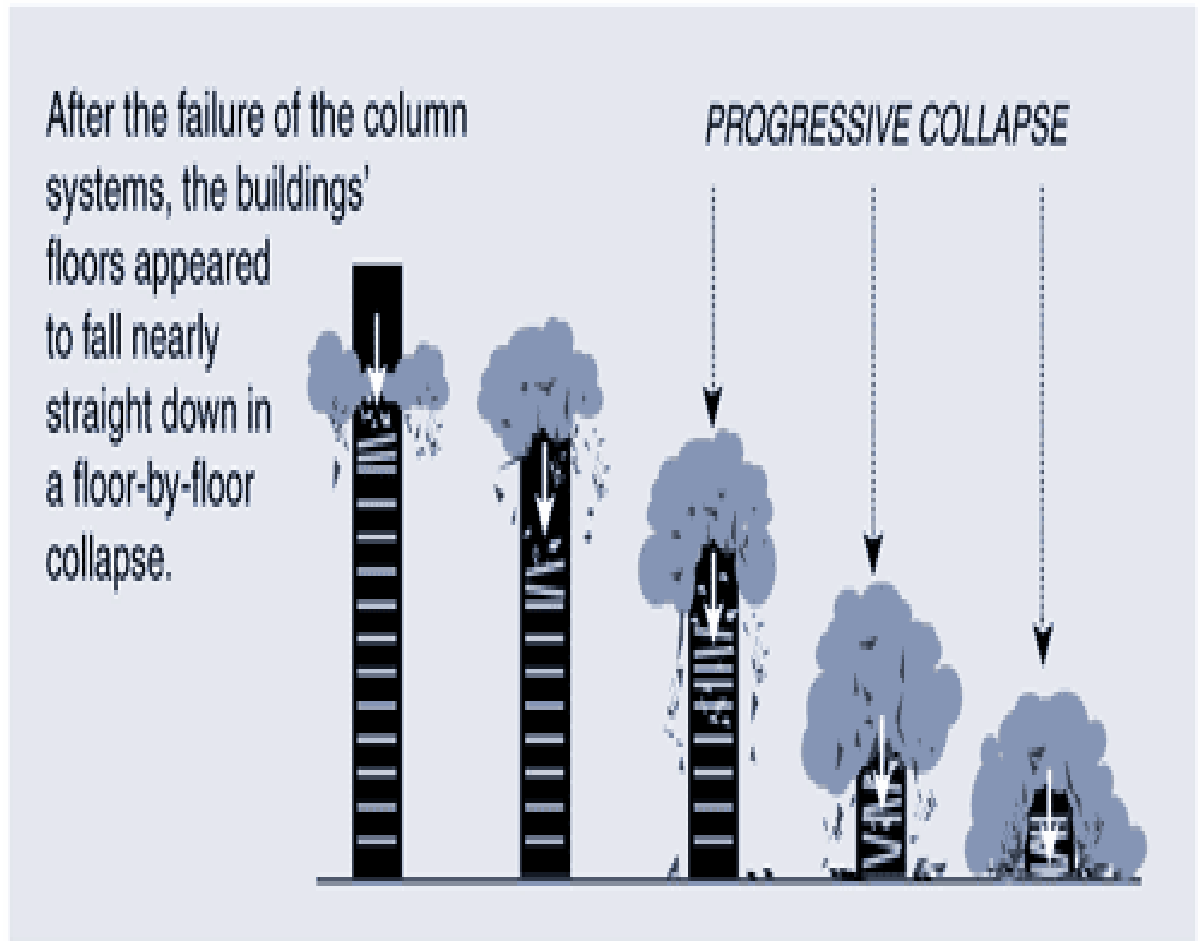


Figure 2.2 – Progressive Collapse for High Rise Building

After the planes crashed with the towers, upper storeys were seriously damaged and several columns were collapsed by the abnormal load. The initial failure triggered a cascade of failure affecting a major portion of the structure and total collapse as shown in **Figure 2.2**. That was a case of progressive collapse. The progressive collapse occurred for a short time. A number of people could not escape within a few minutes. So, it caused many people to be killed.

Investigators for 911 attacks reported that progressive collapse was main factor to cause many peoples were killed. Initial collapse (local failure) caused hundreds of people were to be killed but progressive collapse caused (thousands) people to be killed. The World Trade Center tower I –WTC1 was totally collapsed within 20 minutes after the terrorist plane the building which was not enough time to escape from site for total 17400 occupants. The 911 review (2004) said:

“Progressive collapse describes a collapse in which an initiating event leads to a disproportionate collapse. This phenomenon is rare, especially in steel-framed buildings. The phenomenon of total progressive collapse is even rarer. In fact, there appears to be no example of total progressive collapse of a steel-framed building outside of the alleged examples of the Twin Towers and Building 7.” (911 review, 2004)

The National Institute of Standard and Technology – NIST ⁽ⁱ⁾ was employed to investigate and find out the following issues:-

1. The procedures and practices used in the fire resistance design of structures should be enhanced by requiring an objective that uncontrolled fires result in burnout without local or progressive collapse.

^{i.)} NIST is a non regulatory agency of the U.S. Department of Commerce. The purposes of NIST investigations are to improve the safety and structural integrity of buildings in the United States and the focus is on fact finding. NIST investigative teams are required to assess building performance and emergency response and evacuation procedures in the wake of any building failure that has resulted in substantial loss of life or that posed significant potential of substantial loss of life. NIST does not have the statutory authority to make findings of fault or negligence by individuals or organizations. Further, no part of any report resulting from a NIST investigation into a building failure or from an investigation under the National Construction Safety Team Act may be used in any suit or action for damages arising out of any matter mentioned in such report (15 USC 281a, as amended by P.L. 107-231).

2. Removal of thermal expansion from the spandrels and equivalent slabs in the tenant area to avoid local buckling that affected convergence but had little influence on progressive collapse initiation
3. This further increased the gravity loads on the core columns. Once the upper building section began to move downwards, the weakened structure in the impact and fire zone was not able to absorb the tremendous energy of the falling building section and progressive collapse ensued.
4. As with WTC 1, once the upper building section began to move downwards, the weakened structure in the impact and fire zone was not able to absorb the tremendous energy of the falling building section and progressive collapse ensued.
5. The downward movement of this structural block was more than the damaged structure could resist, and progressive collapse began.
6. NIST recommends that the fire resistance of structures should be enhanced by requiring a performance objective that uncontrolled building fires result in burnout without local or progressive collapse.

No similar high rise steel structure buildings collapsed from fire in top-down manner before. Many professional teams had begun to investigate the progressive collapse in steel structure. Some of them pointed out that main reason were high temperature to

weaken the steel structure and dead weight of debris from local failure. The collapse led to demands by the public to amend current building codes and provide protection against collapse caused by extreme events. Many Scientific and Engineer were researching the important issues which include assessment of load, analysis methods, and design philosophy etc.

The NIST found that design and approval of The World Trade Center were consistence with the provision of the New York City Building Code at that time. NIST found the fire rating of the floor system to vary between 3/4 hour and 2 hours; in all cases, the floors continued to support the full design load without collapse for over 2 hours. The wind loads governed the structural design of the external columns and provide the baseline capacity of the structures to withstand abnormal events such as major fires or impact damage. It significantly exceeded the requirements of the New York City Building Code. The wind load estimated by independent commercial consultants in 2002 were based on wind tunnel tests and differed by as much as 40 percent. The building codes do not require building design to consider aircraft impact. No experience with a disaster of such magnitude and any collapse of high rise building occurred so rapidly and little warning. In order to improve public safety, NIST had some recommendations as below to Public Officials and Building Owners to determine appropriate performance requirements of those tall buildings: These are especially at high risk due to their iconic status, critical function, or design.

1. **Increased Structural Integrity:** The structural integrity should be improved to mitigate the effects of these hazards by enhancement of standard for estimation of load and design of structural systems. – In next section, it will be reviewed for several design method.

2. **Enhanced Fire Resistance of Structure:** The fire resistance should be enhanced by improving the technical basis for construction classification and standard fire resistance testing methods, use of the “structural frame” approach to fire resisting ratings. Moreover, in-service performance requirements and conformance criteria for spray-applied fire resistive materials had been developing and applying in new all building.
3. **New Method for Fire Resistance Design of Structures:** The Performance based methods are alternative to prescriptive design methods. The development and evaluation of new fire resistive coating materials and technologies become a great duty for Scientist and Engineer. The objective is to enhance procedures and practices by requiring uncontrolled fires result in burnout without local or progressive collapse.
4. **Improved Active Fire Protection:** Any fire protection system should be enhanced through improvement to design, performance, reliability, and redundancy of such systems such as sprinklers, standpipes/hoses, fire alarms and smoke management etc.
5. **Improved Building Evacuation:** The safe and rapid egress facilities can ensure shortest time for escape and better occupant preparedness for evacuation during emergencies.
6. **Improved Procedures and Practices:** The design, construction, maintenance and operation of building should be improved to encourage code compliance by

non-government and quasi-government entities, adoption and application of egress and sprinkler requirements in codes for existing buildings and retention and availability of building documents over the life of building.

7. **Education and Training:** For Fire Protection Engineer, Structural Engineer and Architects, the professional skills of building structure and fire safety should be upgraded through professional education and vocational training. All related disciplines shall be encouraged to research the topic of safety for the structural engineering and fire engineering.

(NIST, 2002)

The next chapter will discuss the loading, structural analysis requirements, and design method and design criteria in several international codes such as Hong Kong Code, British Standard and Australian Standard etc.

3. DESIGN METHOD

3.1 Early Stage of Design Methods against Progressive Collapse

Since 1968, after Ronan Point apartment underwent progressive collapse, certain circulars, general recommendations and notes were issued by the Ministry of Housing & Local Government and Institute of Structural Engineers of United Kingdom to propose amendment for Building Regulations. Finally, the Buildings Regulation (Fifth Amendment) came into force in 1970. Before 1970, the following relevant documents were issued by both the Ministry of Housing & Local Government and Institute of Structural Engineers in UK:

1. Gas Explosions in Load-bearing Brick Structures, 1970.(**N.F Astbury et al.,1970**)
2. Flats Constructed with Precast Concrete Panels. Appraisal and Strengthening of Existing High Blocks: Design of New Blocks

From (1) Circular 62/68, there are two basic methods to prevent from progressive collapse:

- Method A: By providing alternative paths of support to carry the load, assuming the removal of a critical section of the load-bearing walls.
- Method B: By providing a form of construction of such stiffness and continuity so as to ensure the stability of the building against forces liable to damage the load-supporting members.

In order to fulfill the requirement, the design force should be assumed to be a standard static pressure of 5 psi (34.48 kPa). It must be considered as loading case for structural design.

For the Method A above, the critical section could be referred to the key element as defined in previous section 2. The first stage of this recommendation in the design process against progressive collapse is to identify all the key elements of the structure by removing the structural elements one at a time. Once a key element is found, the second stage is to provide an alternative load path by assuming that the key element is removed. By doing so, all the key elements could theoretically be eliminated. In addition, since it is assumed that the structure is initially damaged, the design method would be regarded as Damaged Design.

For the Method B, the so-called forces which are liable to damage the load-supporting members could be referred as the accidental loads as defined in previous section 2. With referring to the Ronan Point event, “The Tribunal regarded this explosion as of normal magnitude”. As a result, the maximum accidental load for domestic buildings was regarded as explosive load. “Hence, the magnitude of the accidental load assuming to be equivalent to a standard static pressure of 5 psi (34.48 kPa) was a result of the recommendations of the Ronan Point Tribunal.” By referring to the design method, the suggestions implied to design the element(s) to *directly* resist the liable damaging force, which is the explosive load, by providing the corresponding stiffness and tie forces. Nevertheless, by designing the members which are liable to be damaged by accidental load to provide a resistance equivalent to a static pressure of 5 psi (34.48 kPa) could probably increase the cost of the building dramatically for those buildings using town gas. In the design point of view, it could be regarded as a possible solution and in case

of an accident, it could greatly reduce the structural damage and hence the probability of progressive collapse. Since this design method against progressive collapse does not allow any structural damage, therefore, it would be regarded as a method of Undamaged Design. (Ministry of Housing & Local Government, 1968)

“In December, 1968, The Institution of Structural Engineers issued certain general recommendations on design against progressive collapse of document number RP/68/01 and a document which were numbered RP/68/02 (Notes for Guidance Which May Assist in the Interpretation of Appendix 1 to Ministry of Housing & Local Government Circular 62/68).” (RP/68/01)

Two sections which were relevant to the Method A & B mentioned above were quoted as follows:

“9 It is necessary to ensure that any local damage to a structure does not spread to other parts of the structure remote from the point of mishap and that the overall stability is not impaired, but it may not be necessary to stiffen all parts of the structure against local damage or collapse in the immediate vicinity specifically requires this to be done.” (RP/68/01)

“2.5 It is not required to design individual floor or wall panels to resist an explosive force of this magnitude, but they may be so designed in order to justify the stability of the structures under Method B.” (RP/68/02)

The first recommendation of the above did not allow the spread of any local damage or collapse from the point of mishap to the others parts of the structure. In other words,

chain reaction of failures is not allowed, otherwise, it would be regarded as progressive collapse as classified in previous section 2. In the later parts of this recommendation, it implied the shortcoming of the Method B which recommended designing the elements of new buildings to resist the explosive load. In the second recommendation of the above, it directly pointed out that it is not required to design individual elements to resist the explosive load of magnitude 5 psi (34.48 kPa) under Method B. It implied that if the stability of the structures was designed to be justified, then local *damage* would be allowed. On one hand, the recommendations tried to make the Method B more practical. On the other hand, they seemed to make it tend to use the design principle of Method A. That is, Damaged Design.

Furthermore, one of the recommendations mentioned another method – ‘Venting’. It is required to provide an escape route to the outside air for the explosive pressure. Actually, door and window are weak point on the structure.

“In all room adequate ‘venting’ is required so as to provide an escape route to the outside air for the explosive pressure. This is provided, generally, by the design of the doors and windows or by the arrangement of insubstantial partitions leading to such doors and windows.” (RP/68/02)

It was quite different to above Method A and B because suggestion for design a weakness on the structure rather than stiffness elements. Besides, the weakness part of the structure was designed for removal in an explosive and the load will be transmitted to another alternative paths. That is similar to Method A but it is different not key element to be considered.

“On April 1, 1970, ‘The Building (Fifth Amendment) Regulations 1970’ came into force. This amendment stems from two recommendations of the Ronan Point Inquiry: ‘Recommendation 43: The Building Regulations should include provisions dealing with progressive collapse. Recommendation 44: A Code of Practice applicable specifically to large concrete panel construction should be prepared and published as a matter of urgency’ (RP/68/02)

The proposed amendments which apply to recommendation 44 will not be considered further but, under recommendation 43, in addition to the requirements of the present regulation D8, which is a functional requirement concerned with the safety of buildings in respect of calculated loads, a new regulation D19 is added which applies to a building having five or more storeys (including basement storeys, if any). Briefly, a building must now be constructed in such a way that if a portion of a structural member (other than a portion satisfying certain load conditions) is removed, the consequent structural failure will be limited to an amount specified.

The important provisions are contained in paragraph 4 and 5 of Regulation D19 as follows:

“4. A building to which the provisions of this regulation apply shall be so constructed that if any portion of any one structural member (other than a portion which satisfied the conditions specified in paragraph (5) of this regulation) were to be removed -

(a) structural failure consequent on that removal would not occur within any storey other than the storey of which that portion forms part, the storey next

above (if any) and the storey next below (if any); and

(b) Any structural failure would be localised within each such storey.

5. *The conditions referred to in paragraph (4) of this regulation are that the portion should be capable of sustaining without structural failure the following loads applied simultaneously:*

(a) The combined dead load and imposed load;

(b) A load of 5 pounds per square inch (34.47 kN/m²) applied to that portion from any direction; and

(c) The load, if any, which would be directly transmitted to that portion by any immediately adjacent part of the building if that part were subjected to a load of 5 pounds per square inch (34.47 kN/m²) applied in the same direction as the load specified in sub-paragraph (b).” (Building Regulation in UK, 1985)

A ‘portion’ of a structural member is the lesser of either:

- (a) The part between adjacent supports or between a support and the end of the member; or
- (b) 2.25 times the height of the portion, which with normal storey heights is about 19 feet (5.79m).

This ‘deemed to satisfy’ clause for paragraph 4(b) is:

“If the area within which structural failure would occur would not exceed 750 sq. ft. (69.68 m²) or 15% of the area of the storey, measured in the horizontal plane, whichever is the less’.” (Building Regulation in UK, 1985)

In paragraph 4 (a) of the above regulation, spread of failure to another storey was not

allowed as chain reaction of failures was also considered as a progressive collapse. In the ‘deemed to satisfy’ clause for paragraph 4 (b), it was trying to provide a guidance on the aspect of ‘disproportion’ and make it become more sensible.

3.2 Building Regulation in UK and British Standard Requirements against Progressive Collapse

For disproportionate collapse from Building Regulation UK, it became approved document in Building Regulations UK 1985 as below statement:

“A3 the building shall be so constructed that in the event of an accident the structure will not be damaged to an extent disproportionate to the cause of the damage.” (Building Regulation in UK, 1985)

This requirement was also used in provision of British Standard as below:

Clause 2.2.2.2.b – All buildings are required for effective horizontal ties such as periphery ties, internally ties to columns and walls

Clause 2.2.2.2.c – The layout of building must be checked to identify any failure of any key elements which will not cause the collapse of more than a limited portion. If such elements are identified and the layout can not be revised to avoid from it, the design should take their important into account.

Clause 2.6.2.1 – Design of key element is required if the building is five or more storeys. It only means available to ensuring a structure's integrity in normal use or capability of surviving accidents. Key elements should be designed, constructed and protected as necessary to prevent removal by accident.

Clause 2.4.3.1- Effects of exceptional loads or localized damage

If it is necessary to consider the effects of excessive loads induced by misuse or accident, the load safety factor should be taken to be 1.05 on the defined loads only to be acting simultaneously.

Clause 2.6.2.1 – Key element must be able to withstand accidental load without collapse. It is designed to be adequate stiffness to resist accidental load or protected by some measures such as bollard, defense wall etc. That key element would be known as *protected key elements*.

For BS5628 - Code of practice for use of masonry. Structural use of unreinforced masonry – clause 20.3, the bollard, walls and retaining earth banks should be provided where there is the possibility of vehicles running into and damaging or removing vital load-bearing members of the structure at the ground floor. In addition, structural failure of any member in any storey excluded protected key element should not cause to any failure of the structure beyond the adjacent storeys or beyond an area within those storeys of 70 m² or 15% by area whichever is less. Protected key element or members are single structural elements on which large parts of the structure rely (i.e. supporting a floor or roof area of more than 70 m² or 15% of the area of the storey, whichever is less).

For British Standard BS8110 – Structural Use of Concrete, it is also enforced in United Kingdom and Hong Kong for structural design of reinforced concrete structure. Some requirements and standard are summary as below:-

- i. Progressive collapse had already been taken account into current code in building design in HK and UK.

Robustness (BS 8110-Clause 2.2.2.2 & 2.6):-

Normal method to ensure robustness provides vertical and horizontal ties which are defined as key members. Structures should be planned and designed so that they are not unreasonably susceptible to the effects of accidents. In particular, situations should be avoided from damage to small areas of the structure or failure of single elements and it may lead to collapse of major parts of the structure.

Unreasonable susceptibility to the effects of accidents may be prevented by the following precautions:-

1. All buildings are capable of safely resisting the notational horizontal design ultimate load as given in (Cl. 3.1.4.2) applied to each floor or roof level simultaneously.
2. All building are provided with effective horizontal ties (Cl 3.12.3) including around the periphery, internally and to columns and walls.
3. For the building 5 or more storeys, checking for layout is to identify any key elements failure of which would cause the collapse of more than a limited portion close to the element in question. If such elements are identified and the layout cannot be revised to avoid them, the design should take importance into account given in (Cl.2.6).
4. For the building 5 or more storeys, any vertical load-bearing element other than key element can be removed without causing the collapse of more than a

limited portion close to the element in question. It can be achieved by provision of ties in accordance with (Cl.3.12.3).

- ii) For 5 or more storeys buildings, key elements should be designed incorporated the reasonable means available to ensuring a structure's integrity in normal use or capability of surviving accident.

Loads on key elements:-

In all cases, element and connection should be capable of withstanding a design ultimate load of 34 kPa from any direction and no partial safety factor applied. A horizontal member or part of horizontal member that lateral support vital to the stability of vertical key element, should also be key element. These loads are applied to area which will be projected area of the member.

Key elements supporting attached building components, which should be capable of supporting the reactions from any attached building components to be subject to a design ultimate loading of 34 kPa.

Design of bridging elements:-

For 5 or more storeys buildings, at each storey in turn, each vertical load bearing element is considered lost in turn. The design should be such that collapse of a significant part of structure will not occur. If catenary action is assumed, allowance should be made for the horizontal reactions necessary for equilibrium.

Lateral support: It may be considered to occur at stiffened section of the wall and partition of mass not less than 100 kg/m².

The wall is capable of resisting a horizontal force of $1.5 \times F_t$ and partition is $0.5 \times F_t$.

Where F_t is lesser of $(20 + 4 \times no.)$ or 60 [kN]

no. - The number of storeys in the structure.

3.3 Design Guideline for Civil Structure in Hong Kong about Progressive Collapse

Highway Structure Design Manual (**Highways Dept of HKSAR, 2006**) stated the following requirements and standard:-

For the railway bridge, the potential loading is very large from a derailed train colliding with the substructure of a bridge crossing a railway track. It is difficult to design a support to withstand such load. Consideration shall be given to alleviating the effects of such collapse. The railway authority shall be consulted for design of bridge across or adjacent to railway tracks.

The best defense is located the support of highway and footbridge away at least 5m from the center line of nearest track. If the site condition is limited, the following precautions shall be observed:-

1. Supports shall not be pin-jointed at both top and bottom.
2. A solid plinth shall be provided around 1000 mm height above rail level with “cut-water shaped ends to deflect derailed trains.
3. If no solid plinth, the bottom of support shall be of “cut-water” shape to deflect derailed train.
4. In case a support is formed by a group of individual columns, the support shall

be designed such that removal of one column in that group. It will not lead to the failure of the support under combination 1 load case of BS5400: Part 2.

The support should be designed to withstand a nominal point load of 1000 kN in case of Highway Bridge and 500 kN in case of footbridge at 1200 mm above the rail level to ensure reasonable robustness. Railway under bridges shall be provided with ballast wall at approaches.

The design of piers of bridge over channels shall include consideration of protection against ship collision. Generally, such protection is costly, and the risk involved shall be analyzed and weighed against possibility of protecting the lives of bridge user by means.

3.4 Building Code in Australia (Australian Standard) about Progressive Collapse

Australian Standard is document enforced by law in Australia. The Standard sets out minimum requirements for structural design and construction of concrete structure and steel structure (for example, AS3600 for concrete structure, AS4100 for steel structure, AS 1170 for loading). The following information is from the text book – *Australian Standards for Civil Engineering Student* (Standard Australia, 1998)

3.4.1 Concrete Structure

Australian Standard AS3600 is guide to structural design for reinforced concrete structure in Australia. For chapter 1-Paragraph 2.1:-

“The aims to structural design are to provide a structure which is durable, serviceable, and has adequate strength while serving its intended function and which also satisfies other relevant requirement such as robustness, ease of construction and economy.” (Standard Australia, 1998)

Although it is not obvious to state the exact requirement against progressive collapse, the concept of robustness shall be considered in structural design for certain function intended. It is similar to British Standard. For chapter 1-Paragraph 2.8:

“Requirement such as fatigue, progressive collapse and any special performance requirements shall be considered where relevant and if significant shall be taken into account in the design of the structure in accordance with the principles of this Standard and appropriate engineering principles.” (Standard Australia, 1998)

Progressive collapse is a factor in Structural Design in accordance with the principles of this Standard. For Chapter 1-Paragraph 3.1.1:

“The design of a structure for stability, strength and serviceability shall take account of the action effects directly arising from the following loads:”
(Standard Australia, 1998)

“(e) Accidental loading, if applicable.” (Standard Australia, 1998)

Accidental and abnormal load are also to be considered where structures are important. For example, vehicle impact load will be occurred for carpark parapet.

Steel Structure: Chapter 2 Paragraph 3.11:

“Requirement other than those listed in Clause 3.1.2, such as differential settlement, progressive collapse and any special performance requirements, shall be considered where relevant and, if significant, shall be taken into account in the design of the structure in accordance with the principles of this Standard and appropriate engineering principles.” (Standard Australia, 1998)

3.4.2 Steel Structure

It is similar to previous concrete structure. Progressive collapse shall also be considered in steel structure.

3.4.3 Design Load

Chapter 5 Paragraph 4.5:

“Braking and horizontal impact in carpark, Braking and horizontal impact forces arising from the movement of vehicles shall be treated as additional live loads and calculated as follows:

$F = m \times V^2 / 2 \times \Delta$ where F = impact or braking force, in Newtons

m = gross mass of the vehicles, in kilograms

V = velocity of the vehicles, in meters per second

Δ = deceleration length, in meters” (Standard Australia,

1998)

3.5 Classification of Design Methods

Some design methods and recommendations are very similar. Selection of design method is dependent on the actual site conditions. The following types can be summarized from above sections:-

1. Design key element to resist any collapse.
2. Provide an escape route to the outside air for the explosive pressure. (i.e. venting)
3. Provision of protection to key elements.
4. Provision of tie with adjacent components over an area of local failure.

They can be classified into two main categories: - ‘Damaged Case’ and ‘Undamaged Case’ design as shown in **Figure 3.1**. Those are determined by key element being damaged or not. For damaged case design, key elements can be damaged or even collapsed. On the contrary, the structure must be designed all structural elements including key elements will not be damaged by accidental load for undamaged case design.

Furthermore, undamaged case design can be divided into two types of design method as shown in **Figure 3.1**. One is *Direct Method* and another is *Indirect Method*. Direct method means that key element is designed to resist accidental load by provision of adequate stiffness. That is classified as method (1). Indirect method means that key element is protected to prevent damage from accidental load directly. That is classified as method (2). Another method is required to design some weakness points on structure – ‘venting’ effect in case of an explosion. That is classified as method (3) and

still as undamaged case design. The reason is that the weakness part which is damaged or removed by accidental load can only be a structural or non-structural element but it is NOT a *key element*. Besides, since ‘venting’ can reduce the explosive load such that it is not necessary to design the key element to provide adequate stiffness to resist explosive load, it is regarded as indirect method. For the design method (4), collapse of key element is allowed in case of accident, and therefore, it is classified as damaged case design. In generally, Method (4) is regarded as provision of *Alternative Load Path*. The principle is similar to “*Jenga*” game (See **Figure 3.2**) that means members are removed but the structure will still be stable.

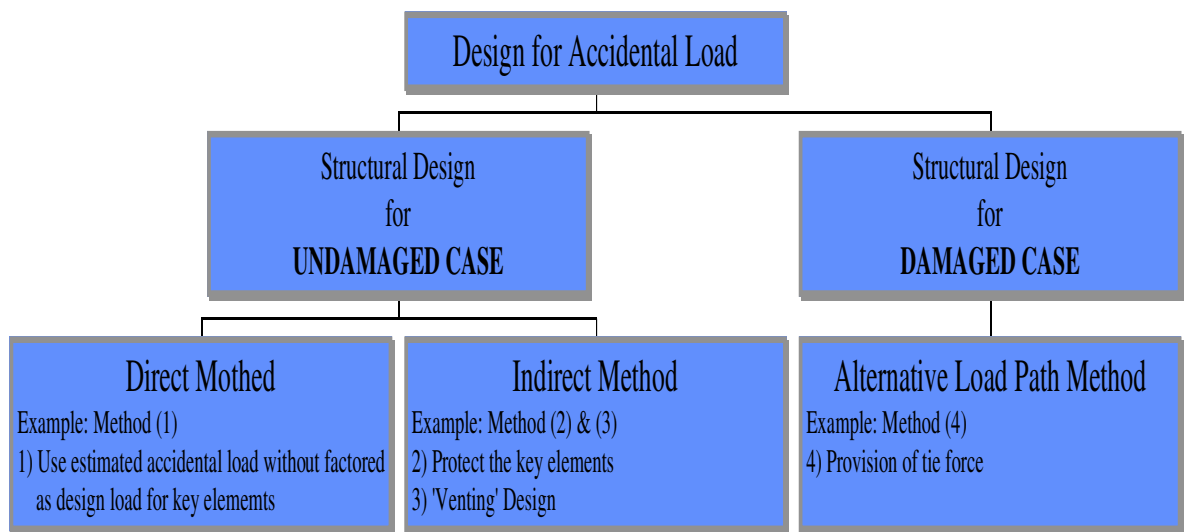


Figure 3.1 - Classification of Design Method against Progressive Collapse



Figure 3.2 – Building Collapse Mode Similar to “Jenga” Game

4. COMPARISON OF DESIGN METHOD

In chapter 3, many design methods were mentioned and classified various types which have their own characteristic. As a structural designer should find out the best solution against progressive collapse, we must recognize each solution for its own merits and shortcomings. Engineer judgment is important for selection of the best method based on case by case in structural design as shown in **Figure 4.1**. The following paragraphs will discuss about each method in details including advantages and disadvantages:-

4.1 Direct Method

It is a basic and simple method against progressive collapse directly. The accidental load must be assumed and higher than service load so much, then the structural elements would be designed to withstand the load with great protection stiffness. It will be very costly and unworthy for construction. Yet, if the accidental load in the real situation is higher than the assumption, the structure will still collapse as same as not adopting the method. The only difference is that money is wasted on strengthening the structure when adopting the direct method.

4.2 Indirect Method

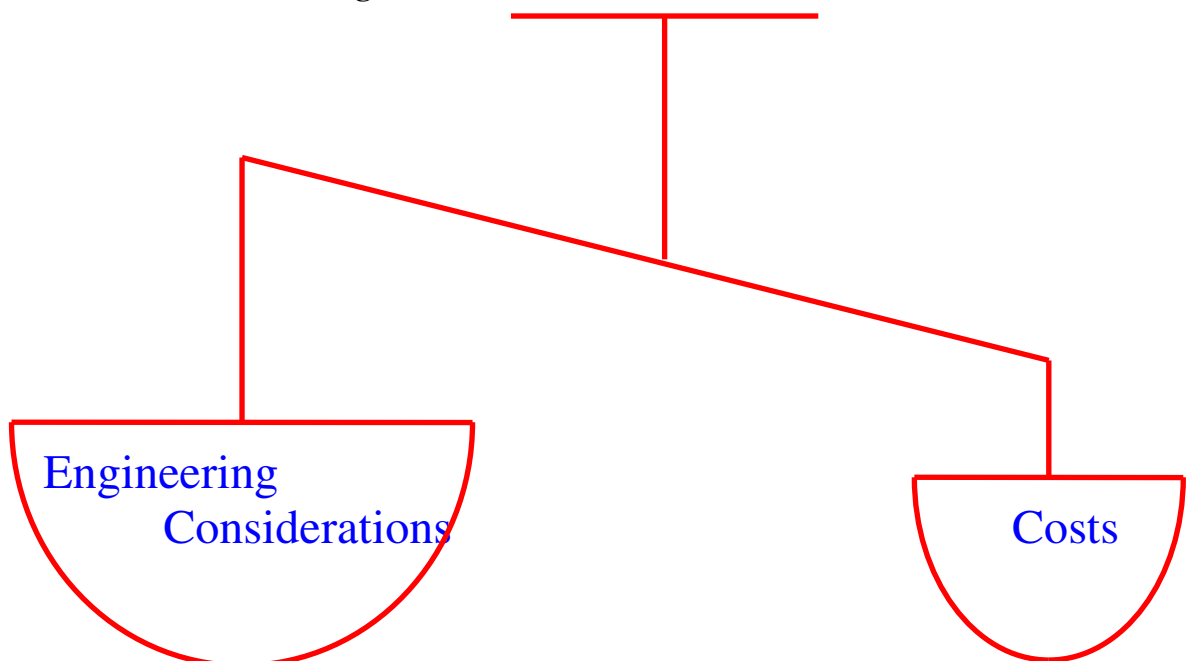
It is quite simple and logical method. The advantage is where cost is cheaper than direct method. It is only required to spent cost for the protecting structure and relatively lower

cost than direct method. Particularly, this is adopted for key elements for ground floor by providing ‘venting’ in case of explosion or vehicle bombing. (For example of bollard as shown in **Figure 4.2**)

4.3 Alternative Load Path Method

This method is assumed a key element which is damaged by accidental load cause the original load path to be changed. It is relatively complicated because it is difficult to predict the possible damaged case. The additional cost could vary depending on the adopted solution (see **Figure 4.1**). However, there are always a number of solutions/schemes which can attain the provision of alternative load path. Furthermore, this method can cope with both accidental and abnormal loading event such that there would have no problem of making decision on whether what kind of accidents and how powerful of the corresponding load should be under considered as the key element is always assumed to be damaged or removed by accidental load or even abnormal load.

Figure 4.1 – Consideration of Cost Factor

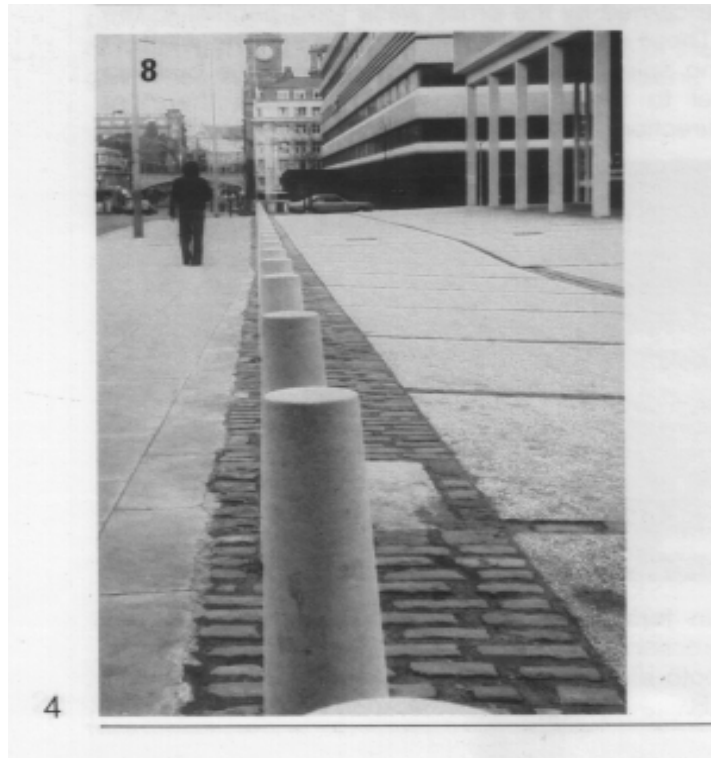


4.4 Comparison for Design Methods

The above design methods will be summary in **Table 4.1**. Direct method is the simplest but it is most costly. Indirect method (for example, bollard) as shown in **Figure 4.2** is cheapest but it is the most limited application. Alternative Load Path Method (i.e. Damaged Design) depends much on the prediction of damaged case while the Direct and Indirect Method (i.e. Undamaged Design) are much more depended on the prediction of accidental event and the corresponding load.

Table 4.1 - Comparison of the Design Methods

Design Method	Abnormal Load	Accidental Load	Applicability	Additional Cost	Degree of difficulty in design
Direct Method	Not considered	Considered	Applicable to most cases	High	Simple
Indirect Method	Not considered	Considered	Limited	Low	Vary
Alternative Load Path Method	Considered	Considered	Applicable to most cases	Vary	Relative difficult



Figures 4.2 - The Columns (Bollard) on the Right Are Being Protected from a Run-way Vehicle.

5. STRUCTURAL ANALYSIS FOR CASE STUDY

In previous chapter, the common design methods against progressive collapse had been mentioned and summarized in **Table 4.1**. It seems that direct method and alternative load path method are applicable to most cases. Alternative load path method may be economical than direct method. In order to verify the applicability of the alternative load path method, a case study of truss beam was introduced and designed by direct method and alternative load path method against progressive collapse. Selection of truss beam is because it is very simple and common in building roof and bridge structure.

For modeling, a truss beam as shown in **Figure 5.1** will be assumed to be a lattice truss of 15 numbers of chord and 2 hinge supports at the both end. One of upper chord at the mid-span was assumed to be removed in damaged design as shown in **Figure 5.2**.

The following cases will be considered, and comparison of the result:

- Case 1 – for normal design by computer program
- Case 2 – for damaged design by hand calculation with “catenary method”
- Case 3 - for damaged design by hand calculation with “impact method” (impact load also considered)
- Case 4 - for damaged design by computer program
- Case 5 – for undamaged design by computer program (that is direct method)

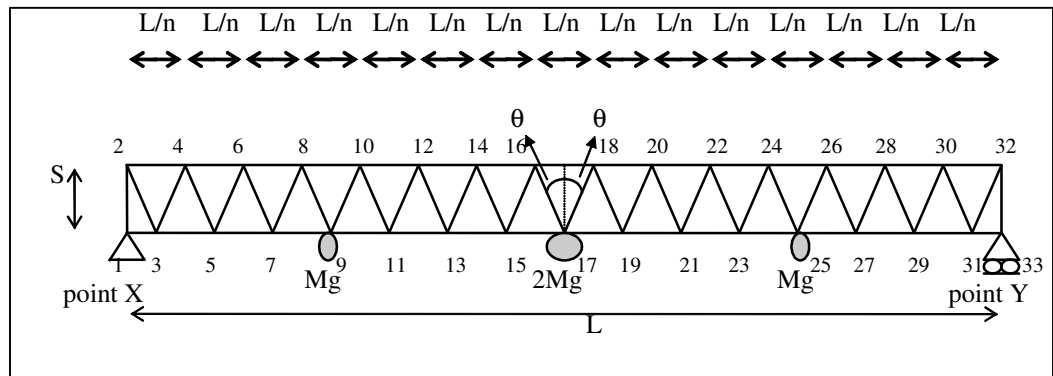


Figure 5.1 - Arrangement of Truss Beam

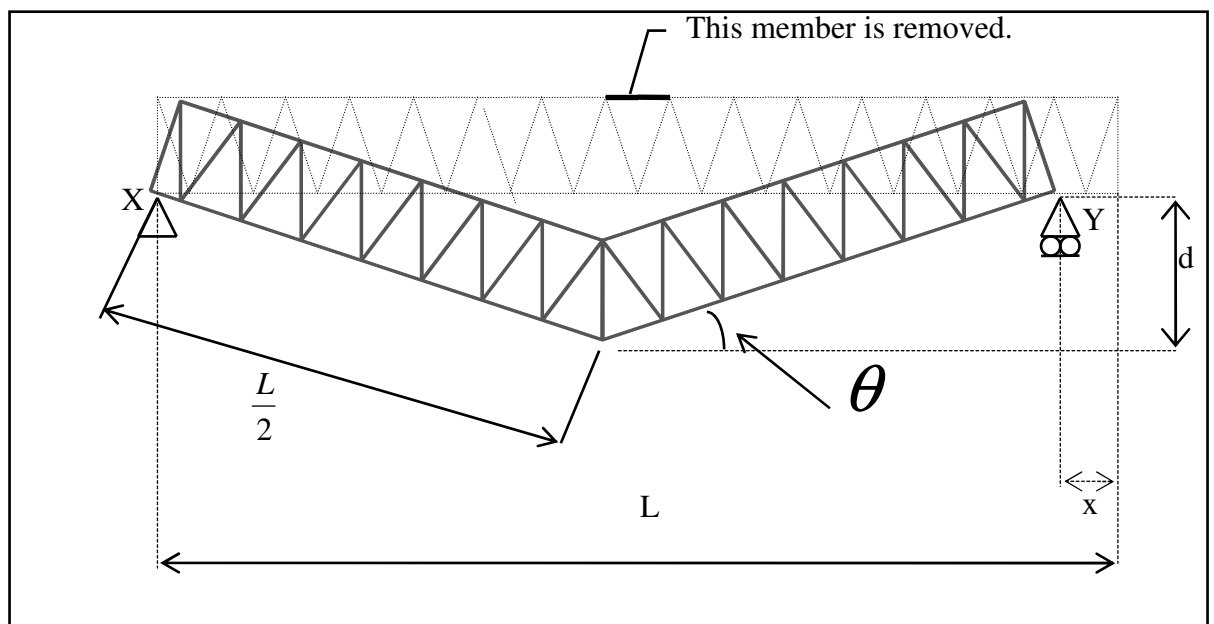


Figure 5.2 - Assumed Failure Mode for Removal of an Element

Legend: d – Vertical displacement

x – Horizontal displacement.

L – Span of structure.

θ - Angle of rotation at collapse.

5.1 Geometrical Design

As shown in **Figure 5.1**, the pin jointed truss was arranged as a normal V-truss with span (L) and depth (S). Two hinge supports were assumed and one will be free for horizontal displacement. The ratio of span and depth was 10: 1. The length of each chord was equal to L/n where n was the total number of horizontal members for the upper chord. Each diagonal brace was formed to be angle of $2 \times \theta$. For the loading, impose load will be considered only owing to design for progressive collapse in this case and dead load from self-weight will be ignored. A certain mass (M) was pre-determined at first and it can be any weight in kg. The truss will withstand 3 point loads – one was 2 times mass (M) at the mid-span and two were mass (M) at one quarter of the span length of beam respectively.

5.2 Procedure for Structural Analysis

For case 1~5, design shall be based on British Standard 5950 – Structural Use of Steelwork in Building (**British Standards Institution, 1990**). The point loads are assumed to be live load and factor of safety to be 1.6 in accordance to British Standard 5950. All steel material is assumed to be grade 43 complying with British Standard 4360. Some assumptions for the structure were made as below:

1. Self-weight to be ignored when comparing with the imposed load.
2. The truss was pin-joint connected and no moment resisting.
3. The truss was simply support with vertical and horizontal restraint which provides the catenary forces on the both sides of the supports.
4. One member (middle upper chord) was being removed for damaged design

but 10 mm clearance was allowed between the corresponding joints.

5. No moment can be transmitted at the both end support.

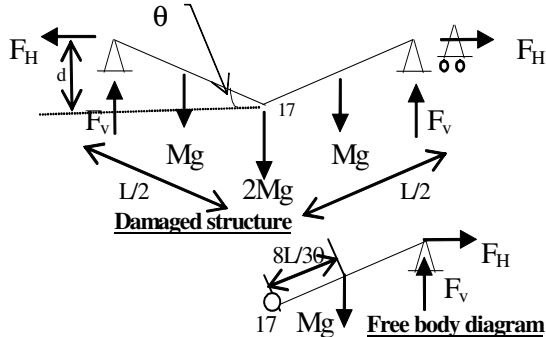
5.2.1 Case 1 – Normal Design for Truss Beam

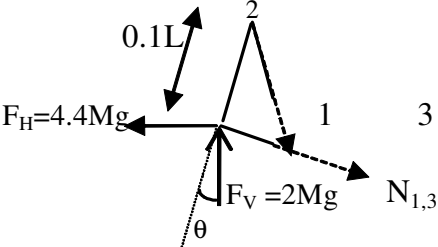
Truss beam as shown in **Figure 5.1** will be analyzed using computer program “*Multiframe*”. In order to match computer program format, mass (M) was assumed to be 1 ton (10 kN) as input data. All node and element of the truss should be label and the restrain conditions and supports should preset in computer program. Moreover, the element section size will be made assumption. The results of axial load for each element and reaction forces were come out in graphical form (for the result as shown in **Appendix B – Figure B.1~B.3**). The maximum member axial force is 73.36 kN (un factored) at middle upper chord which should be the most critical element. Member size of 80 x 80 x 6.3 mm square hollow section is adequate as per calculation in **Appendix B**.

5.2.2 Case 2 – Damaged Design for Truss Beam

Damaged truss beam as shown in **Figure 5.2** using hand calculation will be analyzed with ‘*Catenary Force Method*’ (**Taylor and Schriever, 1976**). For this design method, alternative load path is provided by inducing of catenary force on both sides of the supports such that on removal the member of middle upper chord. When the movement occurs, both of supports will become horizontal restraint to catenary force (F_H) to be stable as shown in next calculation page.

Refer to following calculation sheets for solving for the maximum axial load and reaction forces:

Reference	5.2.2 Case 2 - Catenary Force Design Calculation	Output
<p>Stage 1 – roller at right hand support</p> <p>Stage 2 – to be hinge at right hand support during sliding stop is acting</p> <p>Refer to section 5.2.2</p> <p>Refer to BS5950 page 10, Table 2 for safety factor</p>	<p>i) <u>Simplified analytic model for catenary force calculation</u></p>  <p>Horizontal restraint was provided at joint 16 and 18. Owing to provision of lateral restraint at support 33, the impact force at joint 16 and 18 could be avoided.</p> <p>where F_v = Vertical support F_H = Catenary force L = length of span M = mass of loading g = gravitational acceleration θ = max. angle of rotation at collapse d = max. vertical displacement $\sum F_v$ = Summation of vertical force = 0 $\sum F_H$ = Summation of horizontal force = 0 $\sum M$ = Summation of moment = 0</p> <p>ii) <u>Calculation of Catenary Force</u> Resolving forces vertical: $F_v = 2Mg$</p> <p>$\tan(\theta) = (L/30)/(L/10)$; $\theta = \tan^{-1}(1/3)$; $\theta = 18.4^\circ \Rightarrow \cos \theta = 0.949$</p> <p>$d = \tan(\theta) \times L/2 \times \cos(\theta) = \tan(18.4) \times L/2 \times \cos(18.4) = 0.1581L$,</p> <p>Taking moment at joint 17:</p> $F_H(d) - 2Mg \left(\frac{L}{2} \cos \theta \right) + Mg \left(\frac{8L}{30} \cos \theta \right) = 0$ $F_H(0.1581L) = 0.949MgL - 0.253MgL$ $F_H = 4.4 Mg$ <p>Apply safety factor 1.6 to imposed live loading, Therefore, Factored (F_{DH}) = $1.6 \times 4.4 = 7.0 Mg$</p>	<p>$F_v = 2Mg$</p> <p>Catenary force $F_{DH} = 7.0 Mg$</p>

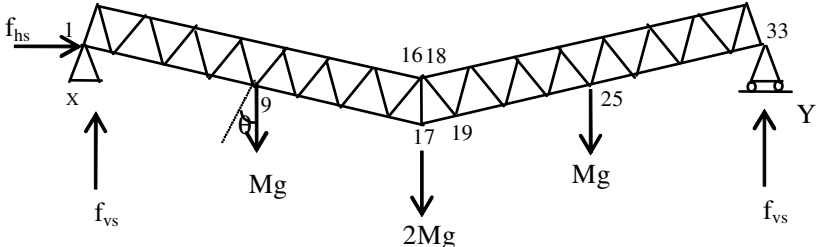
Reference	5.2.2 Case 2 - Catenary Force Design Calculation	Output
	<p>iii) Free body for axial force calculation</p>  <p>iv) Calculation of axial force for member (1,3)</p> <p>Taking moment at joint 2:</p> $N_{1,3}(0.1L) = F_H (0.1L)\cos\theta + F_V (0.1)\sin\theta$ $N_{1,3}(0.1L) = 4.4Mg (0.1L)\cos(18.4) + 2Mg (0.1L)\sin(18.4)$ $N_{1,3} = 7.3 \text{ Mg}$ <p>Apply safety factor of 1.6 to imposed live load, therefore, $F_{DN1,3} = 1.6 \times 7.3 = 11.68Mg$</p>	<p>Critical member force $F_{DN1,3} = 11.68Mg$</p>

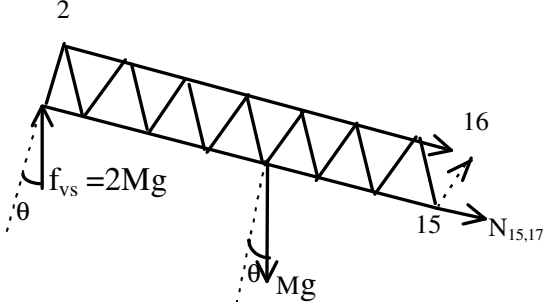
5.2.3 Case 3 – Damaged Design for Truss Beam Considered with Impact Force

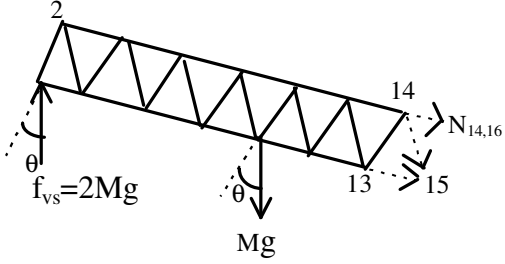
Damaged case truss beam as shown in **Figure 5.2** with hand calculation will be analyzed with '*Impact Design method*' (Taylor and Schriever, 1976). For this design method, it is aimed to the analysis for the dynamic effect on the truss members when two members hit together after removal of middle upper chord member. For stage 1, the truss beam will be analyzed by assuming that it will collapse and attain its equilibrium state under static load according to the damaged case as shown in **Figure 5.2**. The members (14-16) and (15-17) become critical as they carry the maximum load after the truss member (16-18) was removed. The truss beam would be collapsed. For stage 2, the impact will be occurred at joints 16 and 18 and impact force will be added to the damaged structure. The impact load will be calculated and then vertical and horizontal forces at supports can also be calculated. Following assumptions were made as below:

1. At collapse, support Y can provide no frictional or horizontal resistance but vertical resistance only.
2. ω is the angular velocity of the masses immediately before impact of joint 16 and 18.

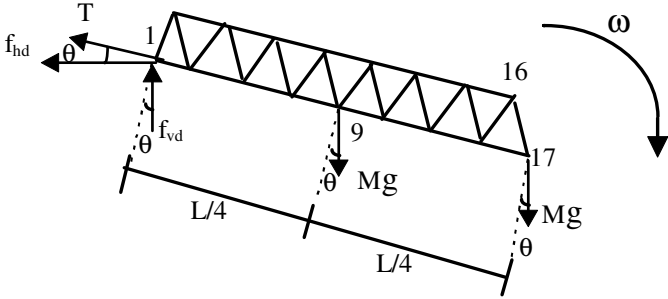
Refer to the following calculation sheets for solving for the maximum axial load and reaction forces:

Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
	<p><u>Stage 1: Calculation of support and axial forces for equilibrium state</u></p> <p><u>(i) Analytic Model</u></p>  <p>where f_{vs} = vertical support reaction when in equilibrium state f_{hs} = horizontal support reaction when in equilibrium state</p> <p><u>(ii) Calculation of vertical and horizontal supports</u></p> <p>Resolving forces vertically:</p> $2f_{vs} = (M + 2M + M)g$ $\therefore f_{vs} = 2Mg$ <p>Taking moment about joint 17:</p> $\Rightarrow f_{hs} = 0$	<p>$f_{vs} = 2 Mg$</p> <p>$f_{hs} = 0$</p>

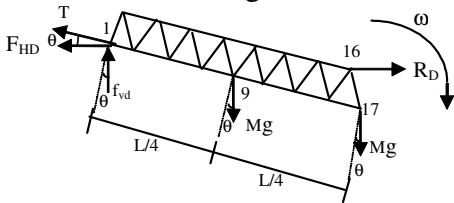
Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
<p>Refer to section 6.1 $\theta = 18.4^\circ$ $\Rightarrow \cos\theta = 0.949$</p>	<p>(iii) Calculation of axial force for member (15,17)</p> <p>Free Body:</p>  <p>Taking moment about joint 16:</p> $f_{vs} \left[\left(\frac{15L}{30} \right) \cos\theta \right] = Mg \left(\frac{8L}{30} \right) \cos\theta + N_{15,17} (0.1L)$ $2Mg \left(\frac{15L}{30} \right) (0.949) = Mg \left(\frac{8L}{30} \right) (0.949) + N_{15,17} (0.1L)$ $\Rightarrow N_{15,17} = 6.96Mg$ <p>Apply safety factor 1.6 on imposed load, then</p> $F_{DN15,17} = 11.14Mg$	<p>$F_{DN15,17}$ $= 11.14Mg$</p>

Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
<p>Refer to section 6.1 $\theta = 18.4^\circ$ $\Rightarrow \cos\theta = 0.949$</p>	<p>(iv) Calculation of axial force for member (14,16)</p> <p>Free Body:</p>  <p>Taking moment about joint 15:</p> $f_{vs} \left(\frac{13L}{30} \right) \cos\theta + N_{14,16} (0.1L) = Mg \left(\frac{6L}{30} \right) \cos\theta$ $2Mg \left(\frac{13L}{30} \right) (0.949) + N_{14,16} (0.1L) = Mg \left(\frac{6L}{30} \right) (0.949)$ $\Rightarrow N_{14,16} = -6.67Mg$ <p>Apply safety factor 1.6 on imposed load, then</p> $F_{DN14,16} = -10.67Mg$	<p>$F_{DN14,16}$ =-10.67Mg</p>

Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
	<p><u>Stage 2: Calculation of supports and axial force for Impact</u></p> <p><u>(A) Calculation of Impact load of joint 16 on 18</u></p> <p>(i) <u>Simplified Analytic Model</u></p> <p>Where L : length of span M : mass of loading θ : angle of rotation at collapse</p>	

Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
	<p>(ii) <u>Immediately before impact of joints 16 on 18</u></p> <p><u>Free Body:</u></p>  <p>where</p> <p>f_{hd} : Horizontal component at support for circular motion f_{vd} : Vertical component at support for circular motion T : Centrifugal force of masses ω : Angular velocity</p> <p>For equilibrium:</p> $T - Mg\sin\theta - Mg\sin\theta = M\left(\frac{L}{4}\right)\omega^2 + M\left(\frac{L}{2}\right)\omega^2$ <p>Energy Eqn:</p> <p>kinetic energy = potential energy</p> $\frac{1}{2}M\left(\frac{L}{4}\right)^2\omega^2 + \frac{1}{2}M\left(\frac{L}{2}\right)^2\omega^2 = Mg\left(\frac{L}{4}\right)\sin\theta + Mg\left(\frac{L}{2}\right)\sin\theta$	<p>eqn(1)</p> <p>eqn(2)</p>

Reference	<u>5.2.3 Case 3 - Impact Design: Detail Calculation</u> Calculation	Output
Refer to section 5.2.2 $\theta = 18.4^\circ$	From eqn(1), $\left(\frac{4}{3L}\right)(T - 2Mg\sin\theta) = M\omega^2$	eqn(3)
	From eqn(2), $M\omega^2 = \frac{24}{5L} Mg\sin\theta$	eqn(4)
	Sub eqn(3) into (4), we have $\frac{3}{4L}(T - 2Mg\sin\theta) = \frac{24}{5L} Mg\sin\theta$	
	$T = \frac{28}{5} Mg\sin\theta$	$T = 1.77Mg$
	$T = 1.77Mg$	
	Vertically Therefore, $f_{vd} = T\sin\theta$ $f_{vd} = \frac{28}{5} Mg\sin^2\theta$ $f_{vd} = 0.56Mg$	$f_{vd} = 0.56Mg$
	$F_v = f_{vsc} + f_{vd}$ $= 2Mg + 0.56Mg$ $F_v = 2.56Mg$ Apply safety factor 1.6 on imposed load, then $F_{DV} = 4.10 Mg$	$F_{DV} = 4.10Mg$

Reference	5.2.3 Case 3 - Impact Design: Detail Calculation Calculation	Output
Axial force for $N_{16,17}$ $= -7.33Mg$	<p>Horizontally, $f_{hd} = T \cos \theta$ $= \frac{14}{5} Mg \sin 2\theta$ $f_{hd} = 1.68Mg$</p> <p>$F_H = f_{hs} + f_{hd}$ $= 0 + \frac{14}{5} Mg \sin 2\theta$ $F_H = 1.68Mg$</p> <p>Apply safety factor 1.6 on imposed load, then $F_{DH} = 2.69 Mg$</p>  <p>Resolving the forces of the free body horizontally: $R_D = F_{HD} = \frac{14}{5} Mg \sin 2\theta$ $R_D = 1.68Mg$</p> <p>Impact load of joint 16 on 18 (R) $= R_D + N_{16,18}$ $R = (-1.68 - 7.33)Mg$ $R = 9.01Mg$ in compression</p> <p>Apply safety factor 1.6 on imposed load, then $F_{DR} = 14.42 Mg$</p>	<p>$f_{hd} = 1.68Mg$</p> <p>$F_{DH} = 2.69Mg$</p> <p>$R_D = 1.68Mg$</p> <p>$F_{DR} = 14.42 Mg$</p>

Member size 90 x 90 x 5 mm square hollow section is adequate as per calculation in

Appendix B.

5.2.4 Case 4 – Damaged Design for Truss Beam with Computer Program

Damage case truss beam as shown in **Figure 5.2** using computer program “*Multiframe*” will be analyzed. The procedure and loading are same as case 1 but the middle upper chord member is deleted and the collapse shape as shown in **Figure 5.2**. The results are shown in **Appendix B, Figure B4~B7**. The maximum member axial force is 74.94 kN (un-factored) at the lower chord at mid-span. Member size 80 x 80 x 6.3 mm square hollow section is also adequate as per calculation in **Appendix B**. The result is slightly higher than case 1 owing to induced additional force by collapse shape.

5.2.5 Case 5 – Undamaged Design by Direct Method for Truss Beam with Computer Program

Undamaged case of truss beam as shown in **Figure 5.1** using computer program “*Multiframe*” will be analyzed. The procedure is same as case 1 but the accident load is added to the structure. The additional force is 34 kN/m^2 acting on the critical member with referring to British Standard BS8110, Part 2 clause 2.6. The critical member should be upper chord at mid-span that is similar to case 1. Based on the assumption of 1 m width of loading area, the line load should be 34 kN/m. The safety factor is 1.0 for accidental load. The results are shown in **Appendix B, Figure B8~B12**. The maximum member axial force is 196.67 kN (factored) at the lower chord at mid-span. Member size 100 x 100 x 6.3 mm square hollow section is also adequate as per calculation in **Appendix B**.

5.2.6 Comparison of the result

The maximum member axial force and reaction forces are summarized and tabulated in **Table 5.1** for case 1 to 5.

Load Case	Design Method	Member Axial Force (factored)	Horizontal Reaction Force at X (factored)	Horizontal Reaction Force at Y (factored)	Vertical Reaction Forces at X,Y (factored)
Case 1 – Normal Structure	“Multiframe” Computer Program	=1.6 x 74 =118 kN	= 0 kN	= 0 kN	=1.6 x 2 Mg =1.6x2x10 =32 kN
Case 2 – Damaged Structure	“Catenary Method”	=11.68Mg =11.68x10 =116.8 kN	= 7.0 x 10 =70 kN	= 7.0 x 10 = 0 kN	=1.6 x 2 Mg =1.6x2x10 =32 kN
Case 3 – Damaged Structure	“Impact Method”	=14.42 Mg =14.42x10 =144.2 kN	= 2.69 x 10 =26.9 kN	= 0 kN	= 4.10 Mg =4.10x10 =41 kN
Case 4 – Damaged Structure	“Multiframe” Computer Program	=1.6x74.94 =120 kN	= 0 kN	= 0 kN	=1.6 x 2 Mg =1.6x2x10 =32 kN
Case 5 – Undamaged Structure	“Multiframe” Computer Program	=196.67 kN	= 0 kN	= 0 kN	=37 kN

Table 5.1 – Comparison of Calculated Result for Cases 1 to 5

Notes:

1. Case 1~4 are based on alternative load path method.
2. Case 5 is based on direct method.

The following findings are based on the result as shown in Table 5.1:

- For the member axial force, case 5 (direct method) is greater approximate 40 % than case 3. That means alternative load path is more economical than direct method. The unit weight of member size 90 x 90 x 5 mm square hollow section is 13.3 kg/m. The unit weight of member size 100 x 100 x 6.3 mm square hollow section is 18.4 kg/m. The weight is greater approximate 40 % with increasing in construction cost by 40 %.
- For the member axial force, case 3 (impact method) is approximate 25 % greater than case 2. If analysis is only considered static load for alternative load path method, certain allowance should be made for dynamic effect for conservative approach. For simplify safety factor, it can be further researched.
- For the member axial force, case 4 (computer application) is nearly same as case 2. Computer software can be applied for alternative load path method

5.2.7 Deficiency of Case Study

In the case 2 design, the provision of the calculated catenary force was based on the static state of the system. The centrifugal force, which was required to maintain the circular motion, was not taken into account. However, this centrifugal force was expected to be smaller than that of calculated in stage 2 (i.e. $T = 1.77Mg$) because the angular velocity was reduced by the provision of catenary force. In general, if the allowable vertical displacement is small, the centrifugal force or the dynamic effect will be small as well. Therefore, it is suggested to make an allowance on the catenary force calculation if required.

In addition, since the joints between truss members were not perfectly hinged, there would be some residual moment which could help to reduce the impact load and the

provision of catenary force. Moreover, it was reasonable to assume that the truss member (16, 18) should be removed because it carried the highest axial force amongst the others. Hence, it was expected to cause the most critical collapse. However, the effect of the removal of each member should be considered in order. Even if the removal of each truss member was taken into account, the design was still not completed because the collapse mode shape of the truss beam was just an assumption. Therefore, the design should be improved by considering other collapse mode shapes. Hence, the design would be lengthy and complicated. Besides, it is not possible to predict all the collapse mode shapes. Therefore, the design can only be made on the possible collapse cases. In future, simplified linear analysis method will be developed to overcome the problem by modification of conventional computer software for structural design. Computer is powerful and useful in tedious process for alternative load path method.

Furthermore, one may argue that the Damaged Design Method only considers the removal of one key element at a time is impracticable as several key elements may be removed at a time. Nevertheless, the objective of the method is to design a structure against progressive collapse which is either disproportionate to initial failure or a chain reaction of failures. In other words, if several key elements were to be removed at a time in an accident, then it would be beyond the scope of the design aspect.

5.2.8 Difficulties of Case Study

Computer software “*Multiframe*”(student version) was applied in the case study. The result was displayed only in graphical form and computer file could not be saved. So, it seems to lack formal report and record for computer output. However, the critical members and forces were only considered. This difficulty was not significant effect.

6. DISCUSSION

As respect to chapter 3 and 4, a number of design methods were generalized as shown in **Figure 3.1**. Those methods were compared and the results were tabulated in **Table 4.1 and 5.1**. According to the damaged design method (i.e. alternative load path method), whenever an accident happens, this design method is expected that the structural elements are to be damaged by the accidental load. Of course, damage of structure is not what everyone expected. Therefore, undamaged design method (i.e. direct method) seems to be more reasonable and preferable but it is costly. However, as it was said that undamaged design is too much depending on the prediction of accidental event and the corresponding load. On one hand, it is quite impossible to say a structure is designed with consideration of all accidental events. On the other hand, the prediction of the corresponding load is another difficult task. Therefore, this design method is not always worked, especially for abnormal loading event. For damaged design method, however, it is not necessary to bother the problem of predicting the accidental loading event. Besides, even if there is an abnormal loading event, the structure may still survived without leading to progressive collapse. Therefore, in case of an accident, structural damage is not preferred but it is better than progressive collapse.

It sounds that damaged design is a radical solution to the problem of progressive collapse. So far, there is generally no consensus among scientists and engineers concerning performance-based requirements for design against progressive collapse. It is because of difficulty and complexity of the collapse analysis of buildings. In future, some simplified linear analysis methods are being developed instead of rigorous nonlinear failure analysis. The computational techniques can be used to track possible

failure mechanism and give useful information to the designer about the integrity of the structure. It can be easily implemented in conventional commercially-available engineering analysis and design software for building structural systems. (Grierson, 2005)

6.1 Risk of Progressive Collapse in Hong Kong

The public suspected that whether current Building Code is enough to ensure protection of public safety against progressive collapse caused by extreme case. In Hong Kong situation, the probability of progressive collapse is very low. British Standards which are enforced in Hong Kong are available for design for progressive collapse. Accidents (i.e. 911 terrorist attack and Ronan Point explosion) had never been occurred before in Hong Kong. However, in future, the probability of terrorist attack and missile attack from Taiwan in a war situation will be increased with danger of progressive collapse of buildings.

In order to enhance the protection to public safety and minimize the construction cost, buildings and civil structures should be classified in accordance to their importance and risk of collapse. Recommendations are given as shown in **Table 6.1**.

<u>Class</u>	<u>Type of Building and Civil Structure</u>	<u>Example</u>	<u>Design Method against Progressive Collapse</u>
Category I	Military Facilities; Nuclear Reactor; Power Station; Large Dam	The headquarters of the People's Liberation Army Hong Kong Garrison (former Prince of Wales Building) – (Figure 10)	Undamaged (direct method) design (extreme case of abnormal and accident load such as Jumbo Jet impact = 5458 kN) ⁽ⁱⁱ⁾
Category II	Symbolical Building and Key Civil Structure and Precast Concrete Building and government building	International Finance Center (Figure 11) ; Tsing Ma Bridge (Figure 12) ; HK International Airport	Undamaged (direct or indirect method) or damaged design (case of abnormal and accident load such as Uniform distributed load = 34 kN/m ²):- e.g. ref. to BS8110 requirements in previous section 4 and bombing attack to be considered.
Category III	For 5 or more storey Building and general civil structure	Residential and Commercial building and infrastructure. (Figure 9)	damaged design (case of abnormal and accident load such as Uniform distributed load = 34 kN/m ²):- e.g. ref. to BS8110 requirements in previous section 4
Category IV	For less than 4 storey Building	Residential and Commercial building etc.	Exempt from checking against progressive collapse

Table 6.1 – Classification of Buildings and Civil Structure

ii) Jet plane impact load: calculation based on Australia Standard AS1170 section 4 – text book (**Standard Australia. 1998**) and assume the deceleration distance (delta) to be 1000 meter.

Mass (m) = 439985 kg (Boeing747 jet); Velocity (V) = 567 km/hr (**wikipedia.2007**)

Impact force (F) = $m \times V^2 / (2 \times \text{delta}) = 439985 \times ((567 \times 1000 / 60 / 60))^2 / (2 \times 1000) = 5458 \text{ kN}$

7. CONCLUSION

The problem of progressive collapse has been discovered for many years. Regulations, Codes and Standards are all available for design. In general, there are two main design principals namely undamaged and damaged design. Both have their own characteristic and advantage to suit for specific case. Undamaged design is based on the prediction of the accidental loading events while damaged design is based on the prediction of the possible damaged cases. However, none of them can be considered as a radical solution to the problems as there are some uncertainties for both of them. Nevertheless, in case of an abnormal loading event, damaged design may be survived from progressive collapse when undamaged design is expected to be failed.

Undamaged design was sub-divided into direct and indirect method. Direct method is simple but costly. Indirect Method has a lower additional cost but there are some limitations on its usage. Alternative load path method is the only method under damaged design. This method is relatively complicated but there are always a number of options to provide an alternative load path. One of the solutions is provision of catenary force but allowance should be given to dynamic effect. In conclusion, damaged design should always be considered as a better method which can be further developed.

For economical point of view and reducing risk of collapse, the level of protection is dependent on the probability of accidental load occurs and the importance of building. This concept is the best solution for balance of safety and cost. If all buildings are designed to able to withstand extreme accidental load (such as Jumbo Jet impact), it is an impossible situation that huge amount of money can not be supported by the society. So, classification of type and function of building is dependent on probability of certain accidental load as presentation in chapter 6.

Category I – very important building and civil structure are designed to withstand extreme accidental load (such as jet impact) by undamaged method. It is not allowed for initial failure.

Category II – key building and civil structure are designed to withstand accidental load (such as bombing attack and 34 kN/m^2) as stated in BS8110 by damaged and undamaged method(direct or indirect method).

Category III – 5 or more storey height buildings or general civil structures are designed to withstand accidental load (such as 34 kN/m^2) as stated in BS8110 by damaged method.

Category IV – Below 5 storey height buildings, these are not specific in requirements to progressive collapse.



Figure 7.1 - The Headquarters of the People's Liberation Army Hong Kong Garrison (Former Prince of Wales Building)



Figure 7.2 - International Finance Centre, (The Tallest Building in Hong Kong)



Figure 7.3 - Tsing Ma Bridge (The World's Sixth Largest Suspension Bridge)



Figure 7.4 - Typical V-Truss Construction for Infrastructure

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Appendix A:-

Project Specification

University of Southern Queensland
Faculty of Engineering and Surveying

ENG 4111/2 Research Project
PROJECT SPECIFICATION

For: Keen Whale, Kwok

Topic: **Understanding progressive collapse and its effect to structural design**

Supervisor: Dr Karu Karunasena

Technical advisor: Ir. Ken Chan (Registered Professional Engineer –Structural Engineering)

Enrolment: ENG 4111 - S1, 2007
ENG 4112 - S2, 2007

Sponsorship: Faculty of Engineering and Surveying, USQ

Project aim: This project seeks to investigate progressive collapse in several causes for building structures, and factors that affect to stabilities, impact and risk to public, and improve in structural design.


Programme: Issue A , 26 March 2007
Issue B , 08 April 2007

1. Research the background information about progressive collapse and structure stability.
2. Cases study for progressive collapse and its impact and risk assessment.
3. Obtain data and information for some cases and analysis data and stability in structural engineering aspect, as appropriate.
4. Evaluate some factors affecting to progressive collapse.
5. Evaluate some structural design to reduce the risk of collapse.
6. Research the effects to public safety and economic efficiency.
7. Analysis and evaluate outcomes and submit dissertation.

As time permits:

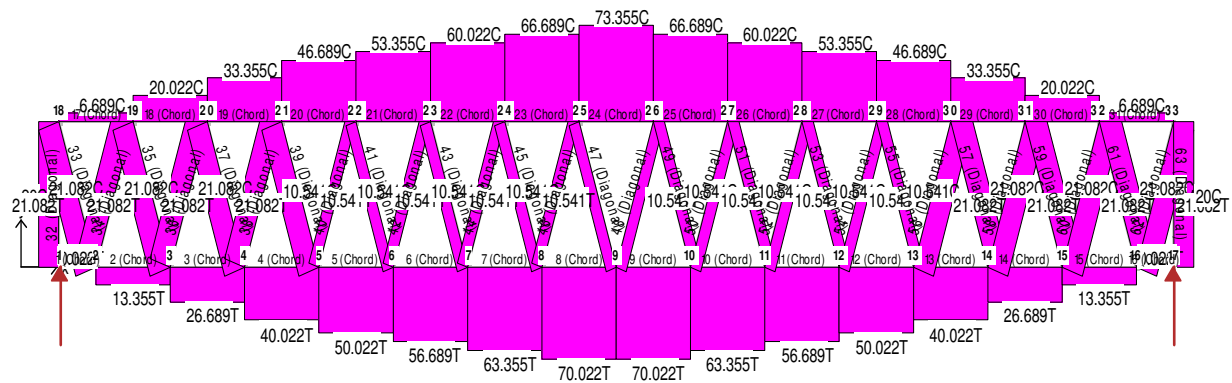
8. Design the practice approach for Structural Engineer for progressive collapse.
9. Suggest progressive collapse to be considered in structural design and impose to current Code of Practice.

AGREED:

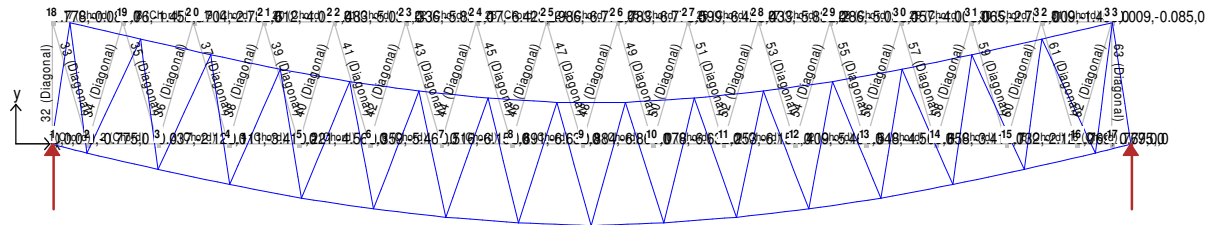
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(date: 06/04/2007) (date: __/__/__)

Appendix B:-

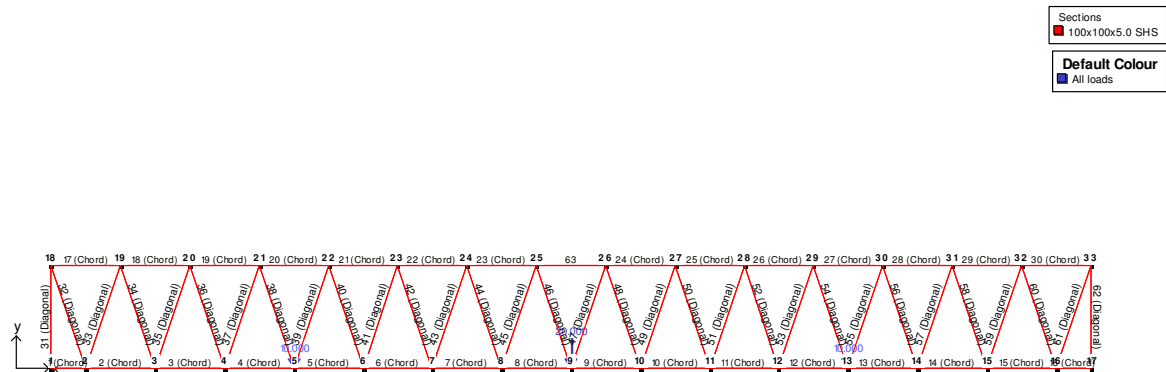
Design Calculation Sheet



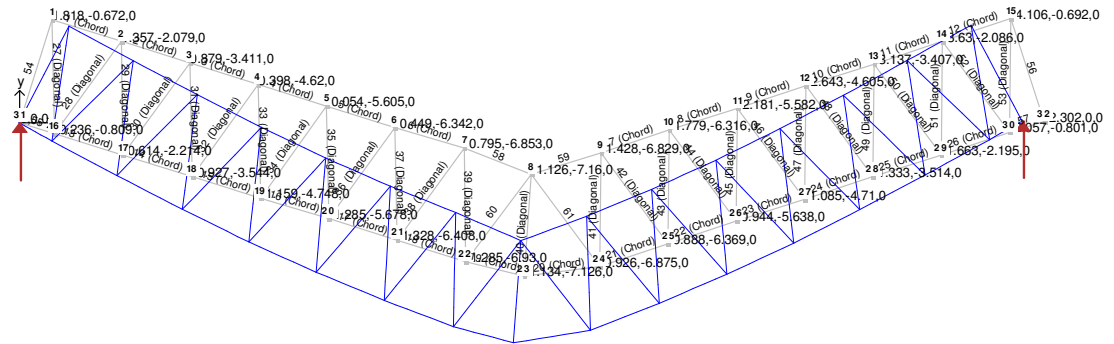
Appendix B: Figure B1 - Case 1 Normal Case for V truss – Member Axial Force



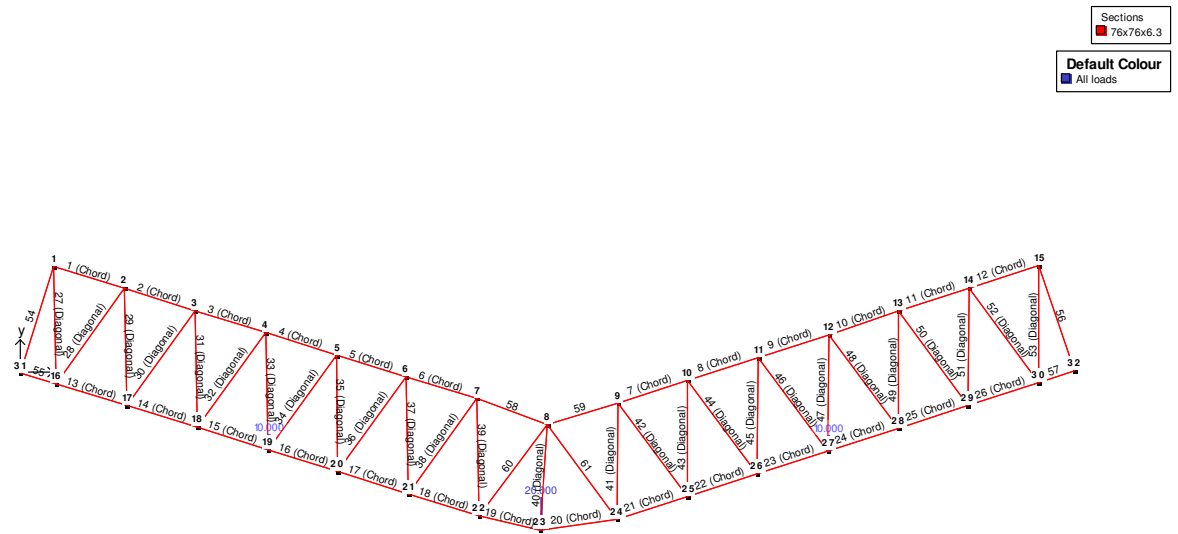
Appendix B: Figure B2 - Case 1 Normal Case for V truss – Deflection



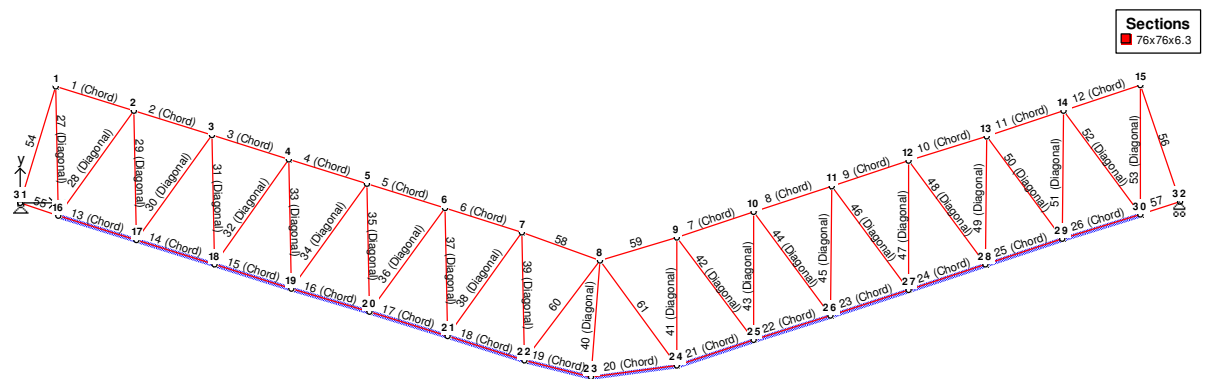




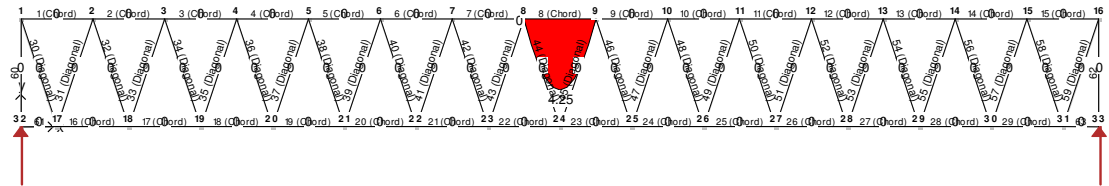
Appendix B: Figure B5 – Case 4 Damaged Case for V Truss Beam – Deflection



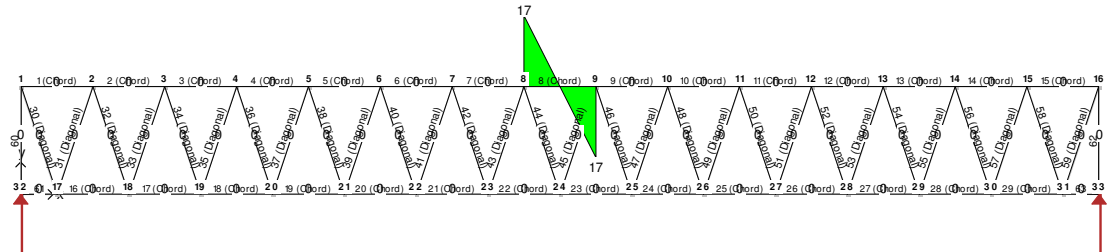
Appendix B: Figure B6 – Case 4 Damaged Case for V Truss Beam – Applied Force



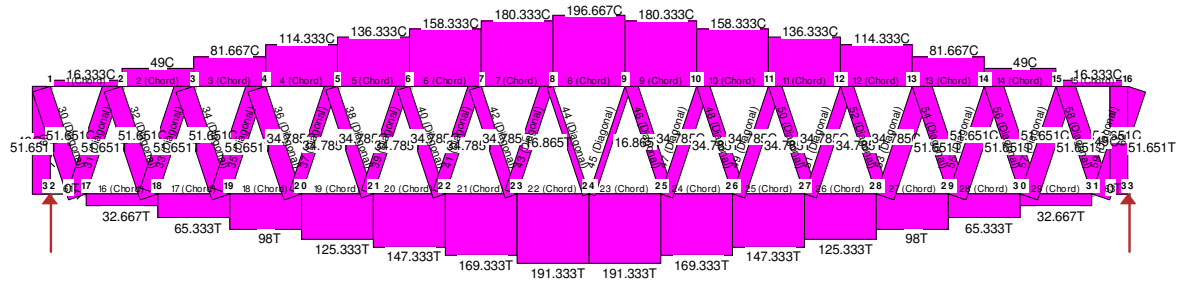
Appendix B: Figure B7 – Case 4 Damaged Case for V Truss Beam – Collapse Shape



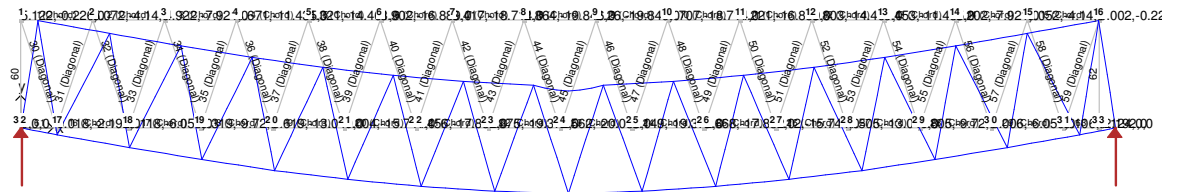
Appendix B: Figure B8 – Case 5 Undamaged Case of V truss beam – Bending Moment Diagram



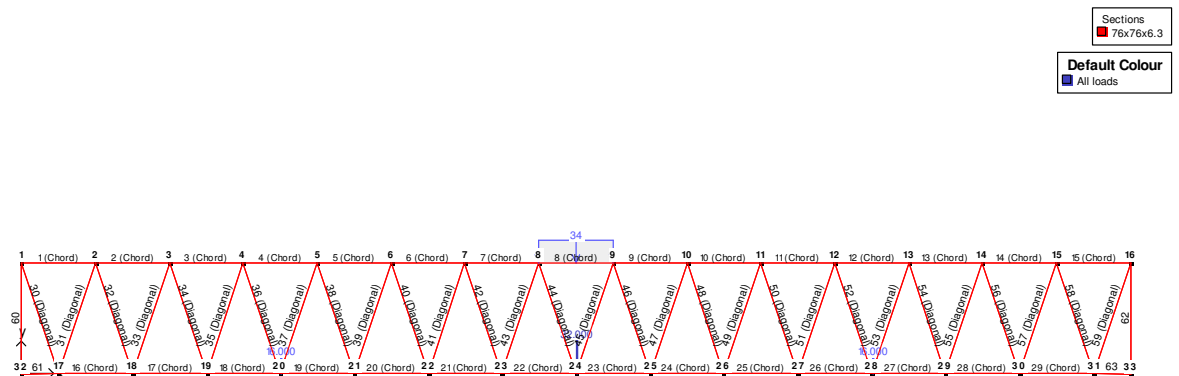
Appendix B: Figure B9 – Case 5 Undamaged Case of V Truss Beam – Shear Force Diagram



Appendix B: Figure B10 – Case 5 Undamaged Case of V Truss Beam – Member Axial Force



Appendix B: Figure B11 – Case 5 Undamaged Case of V Truss Beam – Deflection



Appendix B: Figure B12 – Case 5 Undamaged Case of V Truss Beam – Applied Force

1) Introduction

This document aims to present the design for a V-truss by ultimate state method using computer program “Multiframe” for second order analysis. The truss is formed by mild steel square hollow section (SHS) and the size will be determined. The imposed load is assumed to be 20 kN at mid-span and 10 kN at quarter of span. The truss is designed to withstand the imposed load without overstress and excess deflection. For design axial load of case 1, 3, 4, 5, these were shown in **Table 5.1** of chapter 5.

2) Design Code

- British Standard BS5950 – The Structural Use of Steel

3) Materials

All structural steel shall be Grade 43A complying with BS4360.

4) Design Assumption

- Assume horizontal tie maximum spacing of 5 meter. The effective length (l_e) = 5.0 m

5) Section properties

A) 100 x 100 x 6.3 mm thick. SHS:-

- Inertia (I) = $341 \times 10^4 \text{ mm}^4$
- Section modulus (Z) = $68.2 \times 10^3 \text{ mm}^3$
- Area (A) = $23.4 \times 10^2 \text{ mm}^2$
- Radius of gyration (r_y) = 38.1 mm

B) 80 x 80 x 6.3 mm thick. SHS:-

- Inertia (I) = $165 \times 10^4 \text{ mm}^4$
- Section modulus (Z) = $41.3 \times 10^3 \text{ mm}^3$
- Area (A) = $18.4 \times 10^2 \text{ mm}^2$
- Radius of gyration (r_y) = 30 mm

C) 90 x 90 x 5 mm thick. SHS:-

- Inertia (I) = $202 \times 10^4 \text{ mm}^4$
- Section modulus (Z) = $45 \times 10^3 \text{ mm}^3$
- Area (A) = $16.9 \times 10^2 \text{ mm}^2$
- Radius of gyration (r_y) = 34.6 mm

<u>Input / Reference</u>	<u>Calculation</u>	<u>Output</u>
Refer to Table 5.1 of chapter 5 See attached design table page B12~B16	<u>Truss Diagonal Element for normal – case 1:</u> Design axial load (ND) = 118 kN Compressive strength (Pc) = 121 kN > 118 kN <div style="text-align: right;">OK!</div>	Provide 80x80x6.3 thick SHS
Refer to Table 5.1 of chapter 5 See attached design table page B12~B16	<u>Truss Diagonal Element for normal – case 3:</u> Design axial load (ND) = 144.2 kN Compressive strength (Pc) = 145 kN > 144.2 kN <div style="text-align: right;">OK!</div>	Provide 90x90x5 thick SHS
Refer to Table 5.1 of chapter 5 See attached design table page B12~B16	<u>Truss Diagonal Element for damaged – case 4:</u> Design axial load (ND) = 120 kN Compressive strength (Pc) = 121 kN > 120 kN <div style="text-align: right;">OK!</div>	Provide 80x80x6.3 thick SHS
Refer to Table 5.1 of chapter 5 Using Computer output Appendix B Figure B7 (British Standard Institute, 1990) (The steel Construction Institute, 1990)	<u>Truss Upper Chord Element for Undamaged – case 5:</u> Design axial load (ND) = 196.67 kN Design moment (MD) = 4.25 kNm Compressive strength (Pc) = 239 kN > 196.67 kN <div style="text-align: right;">OK!</div> Moment capacity(Mb) = 22.5 kNm > 4.25 kNm (Buckling resistance) OK! Combined effect = $(196.67/239) + (4.25/22.5) = 1 \leq 1$ <div style="text-align: right;">OK!</div>	Provide 100x100x6.3 thick SHS

Design Tables from Steelwork Design **Guide to BS 5950**

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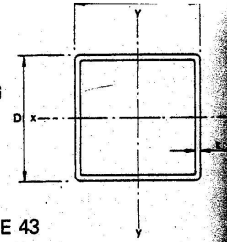
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AXIAL LOAD AND BENDING
SQUARE HOLLOW SECTIONS SUBJECT
TO AXIAL LOAD (COMPRESSION OR TENSION) AND BENDING
LOCAL CAPACITY CHECK



RESISTANCES AND CAPACITIES FOR STEEL GRADE 43

Designation And Capacities	Mass Per Metre (kg)	F/Pz Limit Semi-Compact (Compact)	Moment Capacity M_c (kNm) And Reduced Moment Capacity M_r (kNm) For Ratios Of Axial Load To Axial Load Capacity F/Pz											
			F/Pz	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
70x70x8 Pz = Agpy = 537 pyZ = 9.70	15.3	(1.00)	Mc Mr	11.6 11.6	11.6 11.6	11.6 11.6	11.6 10.9	11.6 9.74	11.6 8.34	11.6 6.84	11.6 5.26	11.6 3.59	11.6 1.84	11.6
80x80x3 Pz = Agpy = 253 pyZ = 6.23	7.22	(1.00)	Mc Mr	7.29 7.29	7.29 7.19	7.29 6.90	7.29 6.42	7.29 5.74	7.29 4.87	7.29 3.93	7.29 2.97	7.29 1.99	7.29 1.00	7.29
80x80x3.6 Pz = Agpy = 301 pyZ = 7.29	8.59	(1.00)	Mc Mr	8.60 8.60	8.60 8.48	8.60 8.14	8.60 7.57	8.60 6.77	8.60 5.75	8.60 4.64	8.60 3.51	8.60 2.36	8.60 1.19	8.60
80x80x5 Pz = Agpy = 409 pyZ = 9.54	11.7	(1.00)	Mc Mr	11.4 11.4	11.4 11.3	11.4 10.9	11.4 10.1	11.4 9.03	11.4 7.68	11.4 6.22	11.4 4.73	11.4 3.19	11.4 1.62	11.4
80x80x6.3 Pz = Agpy = 505 pyZ = 11.4	14.4	(1.00)	Mc Mr	13.6 13.6	13.6 13.6	13.6 13.2	13.6 12.2	13.6 10.9	13.6 9.32	13.6 7.58	13.6 5.78	13.6 3.92	13.6 1.99	13.6
80x80x8 Pz = Agpy = 625 pyZ = 13.4	17.8	(1.00)	Mc Mr	16.0 16.0	16.0 16.0	16.0 15.9	16.0 14.7	16.0 13.2	16.0 11.3	16.0 9.21	16.0 7.06	16.0 4.80	16.0 2.45	16.0
90x90x3.6 Pz = Agpy = 340 pyZ = 9.38	9.27	(1.00)	Mc Mr	11.0 11.0	11.0 10.9	11.0 10.4	11.0 9.69	11.0 8.66	11.0 7.35	11.0 5.93	11.0 4.49	11.0 3.02	11.0 1.52	11.0
90x90x5 Pz = Agpy = 464 pyZ = 12.4	13.3	(1.00)	Mc Mr	14.7 14.7	14.7 14.6	14.7 14.0	14.7 13.0	14.7 11.6	14.7 9.87	14.7 7.99	14.7 6.06	14.7 4.09	14.7 2.07	14.7
90x90x6.3 Pz = Agpy = 575 pyZ = 14.8	16.4	(1.00)	Mc Mr	17.8 17.8	17.8 17.7	17.8 17.0	17.8 15.8	17.8 14.1	17.8 12.0	17.8 9.77	17.8 7.44	17.8 5.03	17.8 2.55	17.8
90x90x8 Pz = Agpy = 713 pyZ = 17.6	20.4	(1.00)	Mc Mr	21.1 21.1	21.1 21.1	21.1 20.6	21.1 19.2	21.1 17.2	21.1 14.6	21.1 11.9	21.1 9.12	21.1 6.19	21.1 3.15	21.1

F = Factored axial load

For compact sections

$M_c = p_y S$ but $\leq 1.2 p_y Z$. Where the value is governed by $1.2 p_y Z$. Higher values of M_c and M_r (for low axial load) may be used if the average factor is greater than 1.2.

For semi-compact sections

$M_c = p_y Z$

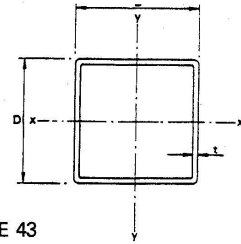
* For slender sections

$M_c = p_y Z$ where p_y is the reduced design strength from Table 8 (BS5950: Part1).

~ Not applicable for semi-compact and slender sections.

FOR EXPLANATION OF TABLES SEE NOTE 4.13.

AXIAL LOAD AND BENDING
SQUARE HOLLOW SECTIONS SUBJECT
TO AXIAL COMPRESSION AND BENDING
OVERALL BUCKLING CHECK



RESISTANCES AND CAPACITIES FOR STEEL GRADE 43

Designation And Capacities	Mass Per Metre (kg)	F/Pz Limit	Compression Resistance Pc(kN) And Buckling Resistance Moment Mb(kNm) For Varying Effective Lengths Le(m) Within The Limiting Value Of F/Pz.														
			Le(m)	1.0	1.5	2.0	2.5	3.0	3.5	4.0	4.5	5.0	5.5	6.0	6.5	7.0	
70x8 = Agpy = 537 Zy = 9.70	15.3	1.00	Pc Mb	508 12.4	468 12.4	399 12.4	310 12.4	234 12.4	179 12.4	140 12.4	113 12.4	92.4 12.4	77.0 12.4	65.2 12.4	55.8 12.4	48.4 12.4	
60x3 = Agpy = 253 Zy = 6.23	7.22	1.00	Pc Mb	245 7.29	233 7.29	215 7.29	188 7.29	154 7.29	123 7.29	98.5 7.29	80.0 7.29	66.0 7.29	55.3 7.29	46.9 7.29	40.3 7.29	34.9 7.29	
60x3.6 = Agpy = 301 Zy = 7.29	8.59	1.00	Pc Mb	291 8.60	277 8.60	255 8.60	222 8.60	181 8.60	144 8.60	115 8.60	93.7 8.60	77.3 8.60	64.7 8.60	54.9 8.60	47.2 8.60	40.9 8.60	
60x5 = Agpy = 409 Zy = 9.54	11.7	1.00	Pc Mb	395 11.5	375 11.5	344 11.5	296 11.5	240 11.5	190 11.5	152 11.5	123 11.5	101 11.5	84.9 11.5	72.0 11.5	61.8 11.5	53.6 11.5	
60x6.3 (80x80x6.3) = Agpy = 505 Zy = 11.4	14.4	1.00	Pc Mb	487 13.9	461 13.9	421 13.9	360 13.9	289 13.9	228 13.9	182 13.9	147 13.9	121 13.9	101 13.9	85.9 13.9	73.7 13.9	63.9 13.9	
60x8 = Agpy = 625 Zy = 13.4	17.8	1.00	Pc Mb	601 16.7	568 16.7	514 16.7	434 16.7	345 16.7	271 16.7	215 16.7	174 16.7	143 16.7	119 16.7	101 16.7	86.9 16.7	75.3 16.7	
60x3.6 = Agpy = 340 Zy = 9.38	9.27	1.00	Pc Mb	332 11.0	319 11.0	301 11.0	274 11.0	237 11.0	197 11.0	161 11.0	132 11.0	110 11.0	92.1 11.0	78.4 11.0	67.4 11.0	58.6 11.0	
60x5 (90x90x5) = Agpy = 464 Zy = 12.4	13.3	1.00	Pc Mb	452 14.7	434 14.7	409 14.7	370 14.7	318 14.7	262 14.7	213 14.7	175 14.7	145 14.7	122 14.7	103 14.7	89.0 14.7	77.3 14.7	
60x6.3 = Agpy = 575 Zy = 14.8	16.4	1.00	Pc Mb	559 18.0	537 18.0	504 18.0	454 18.0	386 18.0	316 18.0	257 18.0	210 18.0	174 18.0	146 18.0	124 18.0	107 18.0	92.8 18.0	
60x8 = Agpy = 713 Zy = 17.6	20.4	1.00	Pc Mb	693 21.8	663 21.8	620 21.8	555 21.8	468 21.8	380 21.8	307 21.8	251 21.8	208 21.8	174 21.8	148 21.8	127 21.8	110 21.8	

Factored axial load

Values of P_c in bold type are for values of slenderness not exceeding 180.

Values of P_c in normal type are for values of slenderness not exceeding 250.

Values of P_c in italic type are for values of slenderness not exceeding 350.

$\phi = \phi S$ (Cl 4.3.7.1.)

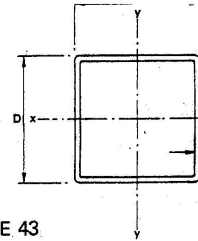
Values have not been given for M_b and P_c where the slenderness exceeds 350.

Note: $M_{bs} = M_b$.

Denotes that section is slender if in axial compression but no allowance has been made in calculating P_c and M_b .

FOR EXPLANATION OF TABLES SEE NOTE 4.13.

AXIAL LOAD AND BENDING
SQUARE HOLLOW SECTIONS SUBJECT
TO AXIAL LOAD (COMPRESSION OR TENSION) AND BENDING
LOCAL CAPACITY CHECK



RESISTANCES AND CAPACITIES FOR STEEL GRADE 43

Designation And Capacities	Mass Per Metre (kg)	F/Pz Limit Semi-Compact (Compact)	Moment Capacity Mc(kNm) And Reduced Moment Capacity Mr(kNm) For Ratios Of Axial Load To Axial Load Capacity F/Pz											
			F/Pz	0.0	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9	1.0
100x100x4 Pz = Agpy = 420 pyZ = 12.9	12	(1.00)	Mc Mr	15.1 15.1	15.1 14.9	15.1 14.3	15.1 13.3	15.1 11.9	15.1 10.1	15.1 8.14	15.1 6.15	15.1 4.14	15.1 2.08	15.1 0
100x100x5 Pz = Agpy = 519 pyZ = 15.6	14.8	(1.00)	Mc Mr	18.4 18.4	18.4 18.2	18.4 17.5	18.4 16.2	18.4 14.5	18.4 12.3	18.4 9.97	18.4 7.56	18.4 5.09	18.4 2.57	18.4 0
100x100x6.3 Pz = Agpy = 644 pyZ = 18.7	18.4	(1.00)	Mc Mr	22.5 22.5	22.5 22.2	22.5 21.3	22.5 19.8	22.5 17.8	22.5 15.1	22.5 12.2	22.5 9.30	22.5 6.28	22.5 3.18	22.5 0
100x100x8 Pz = Agpy = 801 pyZ = 22.4	22.9	(1.00)	Mc Mr	26.9 26.9	26.9 26.9	26.9 26.0	26.9 24.2	26.9 21.6	26.9 18.4	26.9 15.0	26.9 11.4	26.9 7.76	26.9 3.94	26.9 0
100x100x10 Pz = Agpy = 977 pyZ = 26.1	27.9	(1.00)	Mc Mr	31.3 31.3	31.3 31.3	31.3 31.0	31.3 28.8	31.3 25.8	31.3 22.0	31.3 18.0	31.3 13.8	31.3 9.38	31.3 4.79	31.3 0
120x120x5 Pz = Agpy = 629 pyZ = 23.0	18	(1.00)	Mc Mr	27.1 27.1	27.1 26.7	27.1 25.6	27.1 23.8	27.1 21.3	27.1 18.1	27.1 14.6	27.1 11.0	27.1 7.42	27.1 3.74	27.1 0
120x120x6.3 Pz = Agpy = 783 pyZ = 28.0	22.3	(1.00)	Mc Mr	33.3 33.3	33.3 32.8	33.3 31.5	33.3 29.3	33.3 26.2	33.3 22.3	33.3 18.0	33.3 13.6	33.3 9.20	33.3 4.65	33.3 0
120x120x8 Pz = Agpy = 977 pyZ = 33.8	27.9	(1.00)	Mc Mr	40.6 40.6	40.6 40.3	40.6 38.7	40.6 36.0	40.6 32.2	40.6 27.4	40.6 22.2	40.6 16.9	40.6 11.4	40.6 5.79	40.6 0
120x120x10 Pz = Agpy = 1200 pyZ = 39.9	34.2	(1.00)	Mc Mr	47.9 47.9	47.9 47.9	47.9 46.5	47.9 43.2	47.9 38.7	47.9 32.9	47.9 26.8	47.9 20.5	47.9 13.9	47.9 7.06	47.9 0
120x120x12.5 Pz = Agpy = 1460 pyZ = 46.2	41.6	(1.00)	Mc Mr	55.5 55.5	55.5 55.5	55.5 55.2	55.5 51.3	55.5 45.9	55.5 39.2	55.5 32.1	55.5 24.6	55.5 16.8	55.5 8.56	55.5 0

F = Factored axial load

For compact sections

$M_c = p_y S$ but $\leq 1.2 p_y Z$. Where the value is governed by $1.2 p_y Z$, higher values of M_c and M_r (for low axial load) may be used if the average local factor is greater than 1.2.

For semi-compact sections

$M_c = p_y Z$

* For slender sections

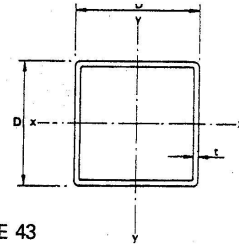
$M_c = p_y r Z$ where $p_y r$ is the reduced design strength from Table 8 (BS5950: Part1).

~ Not applicable for semi-compact and slender sections.

FOR EXPLANATION OF TABLES SEE NOTE 4.13.

AXIAL LOAD AND BENDING
SQUARE HOLLOW SECTIONS SUBJECT
TO AXIAL COMPRESSION AND BENDING

OVERALL BUCKLING CHECK



RESISTANCES AND CAPACITIES FOR STEEL GRADE 43

Designation And Capacities	Mass Per Metre (kg)	F/Pz Limit	Compression Resistance Pc(kN) And Buckling Resistance Moment Mb(kNm) For Varying Effective Lengths Le(m) Within The Limiting Value Of F/Pz.													
			Le(m)	2.0	3.0	4.0	5.0	6.0	7.0	8.0	9.0	10.0	11.0	12.0	13.0	14.0
100x4 = Agpy = 420 Zy = 12.9	12	1.00	Pc Mb	382 15.1	322 15.1	233 15.1	163 15.1	118 15.1	88.3 15.1	68.6 15.1	54.7 15.1	44.7 15.1	37.1 15.1	31.3 15.1	26.8 15.1	—
100x5 = Agpy = 519 Zy = 15.6	14.8	1.00	Pc Mb	471 18.4	394 18.4	283 18.4	197 18.4	142 18.4	107 18.4	83.0 18.4	66.2 18.4	54.0 18.4	44.9 18.4	37.9 18.4	32.4 18.4	—
100x6.3 = Agpy = 644 Zy = 18.7	18.4	1.00	Pc Mb	582 22.5	483 22.5	344 22.5	239 22.5	172 22.5	129 22.5	100 22.5	79.8 22.5	65.1 22.5	54.1 22.5	45.7 22.5	39.1 22.5	—
100x8 = Agpy = 801 Zy = 22.4	22.9	1.00	Pc Mb	721 27.5	591 27.5	416 27.5	287 27.5	206 27.5	155 27.5	120 27.5	95.7 27.5	78.0 27.5	64.9 27.5	54.7 27.5	46.8 27.5	—
100x10 = Agpy = 977 Zy = 26.1	27.9	1.00	Pc Mb	873 32.7	706 32.7	489 32.7	336 32.7	241 32.7	180 32.7	140 32.7	111 32.7	90.9 32.7	75.5 32.7	63.7 32.7	—	—
120x5 = Agpy = 629 Zy = 23.0	18	1.00	Pc Mb	590 27.1	534 27.1	438 27.1	328 27.1	243 27.1	185 27.1	144 27.1	116 27.1	94.6 27.1	78.8 27.1	66.6 27.1	57.1 27.1	49.4 27.1
120x6.3 = Agpy = 783 Zy = 28.0	22.3	1.00	Pc Mb	733 33.3	661 33.3	538 33.3	400 33.3	296 33.3	225 33.3	176 33.3	141 33.3	115 33.3	95.8 33.3	81.0 33.3	69.3 33.3	60.0 33.3
120x8 = Agpy = 977 Zy = 33.8	27.9	1.00	Pc Mb	913 40.9	820 40.9	660 40.9	487 40.9	360 40.9	273 40.9	213 40.9	170 40.9	139 40.9	116 40.9	98.0 40.9	83.9 40.9	72.7 40.9
120x10 = Agpy = 1200 Zy = 39.9	34.2	1.00	Pc Mb	1110 49.1	995 49.1	791 49.1	579 49.1	426 49.1	322 49.1	251 49.1	201 49.1	165 49.1	137 49.1	116 49.1	99.1 49.1	85.7 49.1
120x12.5 = Agpy = 1460 Zy = 46.2	41.6	1.00	Pc Mb	1350 58.2	1200 58.2	935 58.2	678 58.2	497 58.2	375 58.2	292 58.2	234 58.2	191 58.2	159 58.2	134 58.2	115 58.2	99.5 58.2

Factored axial load

Values of Pc in bold type are for values of slenderness not exceeding 180.

Values of Pc in normal type are for values of slenderness not exceeding 250.

Values of Pc in italic type are for values of slenderness not exceeding 350.

$\phi = \phi S$ (Cl 4.3.7.1.)

Values have not been given for Mb and Pc where the slenderness exceeds 350.

Notes: Mbs = Mb.

Notes: denotes that section is slender if in axial compression but no allowance has been made in calculating Pc and Mb.

FOR EXPLANATION OF TABLES SEE NOTE 4.13.